A **cement** and **concrete** industry publication

Concrete Basements

Guidance on the design and construction of in-situ concrete basement structures

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eurocode 2

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Cover photograph: Basement car park during construction - courtesy of Northfield Construction.
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Concrete Basements

Contents

Symbols and abbreviations

Symbol	Definition
A	Parameter to allow for angularity of particles (degrees)
$A_{c,eff}$	Effective area of concrete surrounding reinforcement
A_{ct}	Area of concrete in tension just prior to onset of cracking
A_s	Cross-sectional area of reinforcement
$A_{s,min}$	Area of reinforcement to achieve controlled cracking
$A_{s,req}$	Area of reinforcement required
B	Parameter to allow for grading of sand and gravel (degrees)
b	Breadth
C	Parameter to allow for results of standard penetration tests (degrees)
c	Nominal cover
c'	Cohesion intercept in terms of effective stress
c_u	Undrained shear strength
d	Effective depth
E	Elastic modulus (Young's modulus of elasticity)
E_s	Design value of modulus of elasticity of reinforcing steel
F	Action
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
f_{ct}	Tensile strength
$f_{ct,eff}$	Mean tensile strength of concrete at the time of cracking
f_{ctm}	Mean tensile strength of concrete
f_{cu}	Characteristic compressive cube strength of concrete at 28 days
f_{yk}	Characteristic yield strength of the reinforcement
G	Shear modulus of soil
g_k	Characteristic permanent action per unit length or area
h	Overall thickness/depth
h_0	Notional size
h_{agg}	Maximum aggregate size
h_d	Hydraulic head
I	Moment of inertia
I_r	Parameter used in the calculation of pressure due to surcharge
K	Allowance for creep
K_{0d}	Coefficient of at rest earth pressure
$K_{0d,\beta}$	Coefficient of at rest earth pressure for a retained earth surface inclined at an angle β to the horizontal
$K_{0d,flat}$	Coefficient of at rest earth pressure for flat ground
K_{ad}	Active earth pressure coefficient

Symbol	Definition
K_h	Horizontal earth pressure coefficient = K_{ad} or K_{0d} as appropriate
K_{pd}	Passive earth pressure coefficient
k	Modulus of sub-grade reaction
l	Span
M	Bending moment
N	Blow count from the standard penetration tests (SPT)
NCCI	Non-contradictory complementary information
N'	Blow count from the SPT and corrected for overburden pressure
NDP	Nationally determined parameter
OCR	Over-consolidation ratio
P_d	Design force due to compaction
P_k	Characteristic force due to compaction
Q	Discrete load
q	Load intensity of beam acting downwards
q	Uniform surcharge at ground surface
q_k	Characteristic variable action per unit length or area
R	Restraint factor
$s_{r,max}$	Maximum spacing of cracks
SLS	Serviceability limit state
T_1	Difference between peak temperature of concrete during hydration and ambient temperature, °C
T_2	Long term drop in temperature after concreting, °C
ULS	Ultimate limit state
u	Pore water pressure at depth z
u_0	Basic punching shear perimeter at 2d from column (or loaded area)
V_{Ed}	Design shear action
v	Shear stress
v_{Ed}	Design shear stress
$v_{Rd,c}$	Shear resistance of section not reinforced for shear
w	Deflection of beam
w_k	Crack width
$w_{k,1}$	One of the crack width limits
w_{max}	Crack width limit
x	Depth to neutral axis
z	Depth below ground surface
z_w	Depth of water table below surface
α_c	Coefficient of thermal expansion
α_e	Modular ratio

Symbol	Definition
β	Slope angle of the ground behind a wall (upward positive)
γ	Weight density (of soil)
$\gamma_{C'}$	Partial factor for the effective cohesion
γ_{cu}	Partial factor for the undrained shear strength
γ_F	Partial factor for actions
γ_G	Partial factor for permanent actions, G_k
$\gamma_{G,fav}$	Partial factor for permanent actions when G_k is beneficial
$\gamma_{G,unfav}$	Partial factor for permanent actions when G_k is adverse
$\gamma_{k,f}$	Characteristic value of the density of the fill material
γ_m	Partial factor for a soil parameter (material property)
γ_M	Partial factor for a soil parameter, also accounting for model uncertainty
γ_Q	Partial factor for variable actions, Q_k
γ_R	Partial factor for resistances (soils)
γ_w	Weight density of water
γ_γ	Partial factor for weight density of soil
γ_φ	Partial factor for the angle of shearing resistance (tan φ')
Δc_{dev}	Allowance made in design for deviation of cover
ε_{ca}	Autogenous shrinkage strain
ε_{cd}	Drying shrinkage strain
ε_{cm}	Mean strain in the concrete between cracks
ε_{cr}	Crack-inducing strain in concrete defined as the proportion of restrained strain that is relieved when a crack occurs
ε_{ctu}	Tensile strain capacity of the concrete
ε_r	Restrained strain in concrete
ε_{sm}	Mean strain in reinforcement allowing for shrinkage etc
ρ	Ratio of total area of reinforcement to the gross section in tension
σ_s	Stress in the reinforcement
$\sigma_{v,z}$	Vertical stress
σ'_{ah}	Active horizontal (soil) pressure
σ'_{ph}	Passive horizontal (soil) pressure
σ'_v	Effective vertical stress (in soil)
ϕ	Diameter of bar
φ'	Effective angle of shearing resistance (angle of friction)
$\varphi_{p,k}$ ($\varphi_{cv,k}$)	Characteristic peak (constant volume) angle of shearing resistance
$\varphi_{p,d}$ ($\varphi_{cv,d}$)	Design peak (constant volume) angle of shearing resistance
φ'_{crit}	Critical state angle of shearing resistance
φ'_d	Design angle of shearing resistance
$\varphi'_{d,f}$	Effective angle of shearing resistance of the fill material

Symbol	Definition
φ'_k	Characteristic angle of shearing resistance (= φ_{max} for granular soils: = φ' for clay soils
φ'_{max}	Peak effective angle of shearing resistance
ψ	Factors defining representative actions of variable actions

1. Introduction

Basements are common in many new developments, particularly in urban areas. The reasons for constructing below ground include overcoming planning restrictions on building height, providing car parking, residential, office, retail and storage/archive space, and accommodating plant rooms. Basements provide greater total floor area, thus using land to greater effect.

Successful design requires an understanding of design, construction methods and the resolution of many construction issues. Additionally, the design and construction of basement structures requires an understanding of soil-structure interaction; a complex subject in its own right.

In design terms, basements are water-excluding structures. There are similarities between the design of basements and water-retaining structures, as both have to avoid water penetration. However basements must also deliver the environment and function required by the client and occupant on the inside of the structure.

This guide covers the design and construction of new build reinforced concrete basements and is based on British Standards and Eurocodes, wherever appropriate. As far as possible the terminology used here reflects that of familiar guidance, particularly for the classification of performance levels and types of construction.

The guide has been written for generalist structural engineers who have a basic understanding of soil mechanics. It is assumed that a specialist geotechnical engineer will be consulted on more complex ground problems. In such cases it will generally be necessary to use the services of the specialist from the early stages of the project.

The economic benefits of basements are discussed in other publications[1]. Temporary works are discussed, but their design is not specifically covered. Elements such as embedded contiguous and secant piled walls, commonly used for temporary works and often incorporated into permanent works, are covered in outline but their design is outside the scope of this publication.

This guide does not cover seismic actions nor does it deal with retro-fitting basements into existing structures. Nor does it cover the use of precast walls, walls made using insulating concrete formwork (ICF) or masonry walls, common in shallow domestic basements, these are fully discussed elsewhere[2, 3, 4]. There are many examples of basements constructed in the UK and beyond that provide a collection of case histories. This guide brings together the salient features for design and construction and references a number of documents that should be consulted for further detail.

The driver behind writing this publication is the need for guidance on designing basements to the Eurocodes, which also introduced new terminology. The aim of this guide is to assist designers of concrete basements of modest depth (not exceeding 10 m). It should also prove relevant to designers of other underground structures.

2. Outline of the design process

The key steps in the design process are shown in Table 2.1. Each step will require discussion, judgement, decision making and usually iteration of designs.

Table 2.1
Basement design process.

Step	Description	Chapter
1	Establish the client's requirements, specifically the *grade* of usage of the basement.	3
2	Commission site surveys and exploratory works to establish the site constraints and conditions, and relevant geotechnical parameters.	3
3	The design team considers the brief and site conditions and submits outline proposals based on outline designs and construction methodology indicating the net basement area achievable, the *type* of construction, the *form* of construction and *Tightness Class* assumed. The client should be appraised of the likely environmental conditions in use.	3 to 11
4	On approval by the client, undertake detailed design comprising selection of materials, establishing preferred method of construction, loads on the basement structure including earth and water pressure during construction and in use, structural design for strength and serviceability, and production of drawings and specifications.	4 to 11
5	Construction.	11

Primary references for the design and construction of basements include:

- Building Regulations[5]
- BS EN 1990: *Basis of structural design*[6]
- BS EN 1991-1-1: *General actions – Densities, self-weight, imposed loads for buildings*[7]
- BS EN 1991-1-6: *Actions on structures during execution*[8]
- BS EN 1992-1-1, Eurocode 2: *Design of concrete structures, Part 1-1: General rules and rules for buildings*[9]
- BS EN 1992-3, Eurocode 2: *Design of concrete structures, Part 3: Liquid retaining and containing structures*[10]
- BS EN 1997-1, Eurocode 7: *Geotechnical design, Part 1: General rules*[11]
- BS EN 206-1, *Concrete, Part 1: Specification, performance, production and conformity*[12]
- BS 8500 Parts 1 & 2: *Concrete – Complementary British Standard to BS EN 206-1*[13, 14]
- BS 8002: *Code of practice for earth retaining structures*[15]
- BS 8102: *Code of practice for protection of structures against water from the ground.* [16]
- CDM Regulations[17]
- CIRIA. Report C660, *Early–age thermal crack control in concrete*[18]. (This document is quoted as NCCI in the UK National Annex to BS EN 1992-3.)

Additional references are noted at the end of this document.

3. Planning of basements

Design of basements requires a collaborative effort from a number of parties, including the client. In addition to the basic design team comprising architect, structural engineer and services engineer, a geotechnical specialist may be required in complex projects. For instance:

- where the soil conditions are unusual and not familiar to the structural engineer;
- where the site has compressible soils where heave is a risk;
- where the basement adjoins sensitive or historic structures or tunnels; or
- when movements need to be predicted.

If a contractor is appointed early (as may happen in design and build contracts) their contribution will be invaluable, particularly as regards methods of construction and temporary works. In the absence of a contractor, the structural engineer will be required to advise on these aspects.

For each aspect of basement design a number of options will usually be available, resulting in a matrix of possibilities for design. The design team will need to assess and synthesise these into a working solution to satisfy the brief.

The design team should be clear on the expectations of the client, who should be made aware of the risks and limitations of the solution and of maintenance implications. Some of the basic terminology is outlined in Table 3.1. This table also indicates where responsibility usually lies for decision making. Guidance for the client may be found in *Reducing the risk of leaking substructure*[19].

Table 3.1
Terminology and decision makers.

Term	Reference	Nomenclature		Decision maker	See Section
Grade	BS 8102[16]	*Grade* 1 2 3 (4)#	Basic Better utility Habitable (Special)	Client	3.2
Type	BS 8102	*Type* A B C	Barrier protection Structurally integral protection Drained cavity protection	Design team	3.3
Form	Various e.g. CIRIA Report R140[20]	Forms of construction, which for reinforced concrete construction include slabs, walls, beams, piles, diaphragm walls, capping beams etc. Viable forms of construction will depend on method of construction		Design team	3.4
Tightness Class	BS EN 1992-3[10]*	*Tightness Class* 0 1 2 3	Some degree of leakage acceptable Limited leakage: any cracks should heal Leakage minimal: staining acceptable. No leakage: special measures (e.g. liners) needed	Structural Engineer	9.6

Key
\# The previous versions of BS 8102, BS 8102:1990, made reference to a Grade 4 (archive storage) and although no longer used, this designation retained here for ease of reference.
* BS EN 1992-3 is for water-retaining structures and the descriptions are not necessarily directly applicable to water-resisting structures.

A number of tasks associated with the achievement of watertightness of basements could be carried out by either the architect or the structural engineer or indeed another party such as the contractor or specialist sub-contractor. It will therefore be prudent to define the responsibilities of each member of the design team in relation to these issues clearly at the outset, and to notify the client accordingly. In large projects, the need for a resident engineer should be discussed with the client.

Generally, watertightness will form part of the structural engineer's brief only when the construction method used is Classified as Type B, 'structurally integral protection' (see Section 3.2). Details of membranes, damp-proofing, tanking and similar measures, may be in, but will usually be outside, the structural engineer's responsibilities. Similarly, responsibility for drainage will need clarifying, as this could be handled by the architect, the structural engineer or the services engineer.

Key issues to be considered are listed below:

- Integral protection
- Damp-proofing
- Tanking
- Drainage
- Ventilation
- Heating/thermal requirements
- Access.

3.1 Use and required environment: *grades* of basements

It is critical that the client states the intended use of the basement space and whether flexibility is required to allow for potential change of use in the future. The final brief from the client to the design team will usually evolve through an interactive process of consultation between the client and the design team. The colloquial term 'waterproof' basement is best avoided. Instead, tolerable degrees of water penetration and vapour penetration should be agreed – see Table 3.2. This will dictate the level of active and passive measures required to control the internal environment.

BS 8102[16] provides a helpful table of Classification and defines *grades* of use. The different *grades* aim to distinguish the different performance levels in a qualitative manner. By way of explanation, Table 3.2 reproduces this information combined with guidance from CIRIA Report R140 (*Water-resisting basements*)[20] which gives guidance on defining the internal environment (temperature, humidity, dampness) for different uses within each *grade* of basement. Figure 3.1 illustrates how different grades might appear once finished.

It should be noted that within a basement, Relative Humidity (RH) is determined by external and internal conditions and controlled internally by natural or mechanical ventilation. In most cases it is not affected by waterproofing measures. The control of RH should be discussed with the design team and client and a strategy agreed. The temperature levels indicated are achieved through heating and insulation. As with RH, they are not affected by the waterproofing measures and therefore no longer a requirement of BS 8102. Special environments such as archive or retail storage can only be achieved with a

heating/ventilation regime along with the appropriate type of construction. In the case of archive storage, appropriate guidance may be found in BS 5454[21].

Table 3.2 illustrates performance standards often required of basements. The ranges indicated for relative humidity and temperature are for guidance only. Quantification of acceptable levels of 'damp patches' is very difficult and not attempted here. Small amounts of water vapour may go through uncracked concrete but this may be controlled by the use of temperature and/or ventilation or membranes.

Grade of basement*	Usage*	Performance level*	Performance level**			
			Relative humidity**	Dampness	Wetness	Temperature
1 (Basic utility)	Car parking Plant rooms (excluding electrical equipment) Workshops	Some leakage and damp areas tolerable. Local drainage may be required	> 65% normal UK external range	Visible damp patches may be acceptable	Minor seepage may be acceptable	Car parks: atmospheric Workshops: 15–29°C Mechanical plant rooms: 32°C max. at ceiling level
2 (Better utility)	Workshops and plant rooms requiring drier environment than Grade 1 Retail storage	No water penetration but damp areas tolerable*** dependent on the intended use. Ventilation may be required to control condensation	35–50%	No visible damp patches***, construction fabric to contain less than air dry moisture content	None acceptable	Retail storage: 15°C max. Electrical plant rooms: 42°C max.
3 (Habitable)	Ventilated residential and commercial areas including offices restaurants etc. Leisure centres	Dry environment. No water penetration. Additional ventilation, dehumidification or air conditioning appropriate to intended use	40–60% 55–60% for restaurants in summer	None acceptable. Active measures to control internal humidity may be necessary	None acceptable	Offices: 21–25°C Residential: 18–22°C Leisure centres - Spectators: 18°C - Squash courts: 10°C - Changing rooms: 22°C - Swimming pools: 24–29°C Restaurants: 18–25°C Kitchens: 29°C max.
(4)** (Special)**	Archives Landmark buildings and stores requiring controlled environment	Totally dry environment. Requires ventilation, dehumidification or air conditioning appropriate to intended use	50% for art storage >40% for microfilms and tapes 35% for books	Active measures to control internal humidity probably essential	None acceptable	Art storage: 18–22°C Book archives: 13–18°C

Key

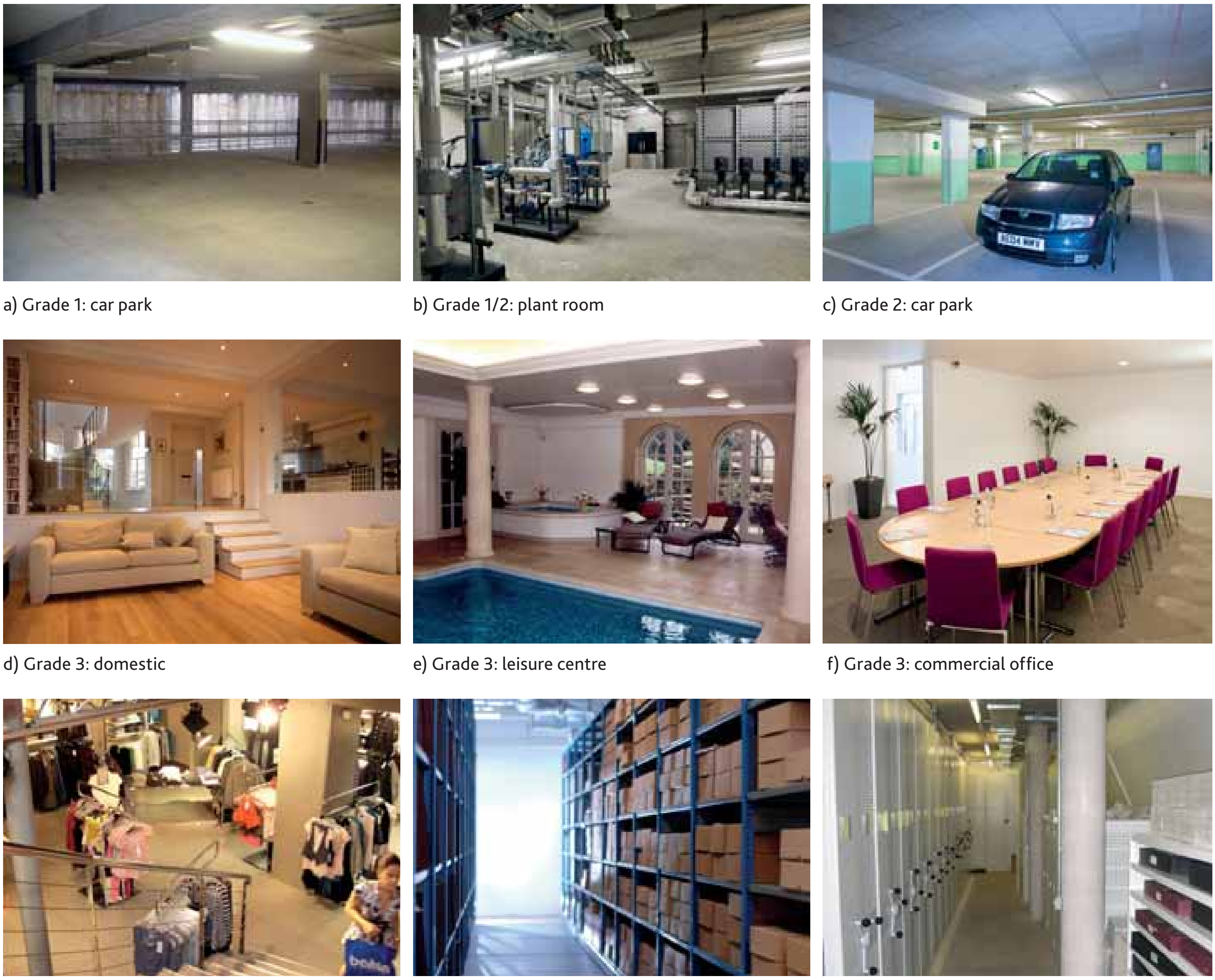
* Based on Table 2 of BS 8102[16].

** Based on Table 2.2 of CIRIA Report R140[20].

*** A damp area is defined under BS 8102 as an area which, when touched, might leave a light film of moisture on the hand but no droplets of water (i.e. beading). 'Damp areas tolerable' may be considered to be inconsistent with 'no visible damp patches' to CIRIA R140. Where needed, clarification of expectation should be sought from the client.

**** The previous version of BS 8102, BS 8102:1990, made reference to a Grade 4 (archive storage) where the only difference from Grade 3 was the performance level related to ventilation, dehumidification or air conditioning. Although no longer used, this designation retained here for ease of reference. BS 8102:2009 makes reference to BS 5454[21] for archive storage.

Table 3.2
Guide to *grades* of basements: functional environmental requirements and levels of protection



a) Grade 1: car park
b) Grade 1/2: plant room
c) Grade 2: car park
d) Grade 3: domestic
e) Grade 3: leisure centre
f) Grade 3: commercial office
g) Grade 3: commercial retail
h) Grade 3: commercial storage
i) Grade (4): archive

Figure 3.1
Typical examples of basement *grades* and usage
Photos: Arup, Durkan Pudelek Ltd, St Martins-in-the-Field, Giles Rochell Photography, Christopher Hill

3.2 *Types* of water-resisting construction/protection

Having established the *grade* of basement required, the main consideration is then to determine the appropriate *type* of construction. BS 8102[16] identifies three types of water-resisting construction/protection, namely, *Types* A, B and C. As explained below, these are, respectively, barrier, structurally integral and drained protection.

Table 3.3 illustrates the acceptability of Types A, B and C with respect to risk and water table Classification. The position of the water table is considered critical to the potential risks to the final construction. Potentially either Type A, B or C may prove acceptable with all water table levels for all grades of basement. However, it will be noted that in variable or high water tables, additional measures are required for Type A and piled wall construction. It will also be noted that risk decreases with reduced permeability of the external ground (where undisturbed) and main structural wall.

Water table Classification[a]		Types of water-resisting construction			
		Type A (barrier protection)	Type B (structurally integral protection)		Type C (drained protection)
			Piled wall	RC wall to BS EN 1992	
Risk associated with water table: Low	Low	Acceptable	Acceptable	Acceptable	Acceptable
↕	Variable	May be acceptable if 'variable' Classification is due to surface water: seek manufacturers' advice	Acceptable where a) the piled wall is directly accessible for repair and maintenance from inside the structure or b) the piled wall is combined with a fully bonded waterproofing membrane or c) the piled wall is faced with a concrete wall to BS EN 1992	Acceptable	Acceptable
High	High	May prove acceptable where a concrete wall to BS EN 1992 is used or where cementitious renders or coatings are used		Acceptable	Acceptable

Notes
Based on Table 1 of BS 8102:2009[16].
Measures to reduce risk

- Combinations of types should be considered when risks or the consequences of failure are high or where additional vapour checks are required
- Appropriately designed and maintained sub-surface drainage may be used to lower risk
- A fully bonded waterproofing membrane may be used to lower risk
- Waterproofing admixtures and appropriate supervision lowers risk
- Risk decreases with reduced permeability of the main structural wall
- Discharge systems (e.g. pumps) must be maintained for the system to remain effective.

Key
a Water table Classification:
Low = where the water table is assessed to be permanently *below* the base slab (free draining strata only).
High = where the water table is assessed to be permanently *above* the underside of the base slab.
Variable = water table fluctuates.

Table 3.3
***Types* of water-resisting construction and acceptability**

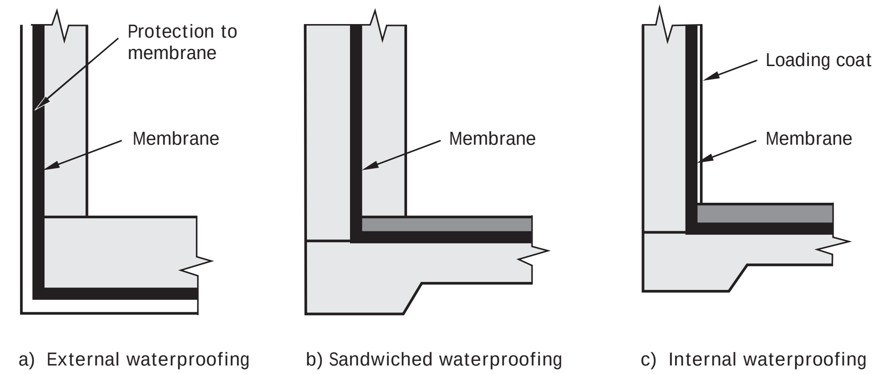
Type A – barrier protection

As shown in Figure 3.2, this type of construction relies entirely on a continuous barrier of a waterproofing membrane, which can be on the exterior faces of walls and floors, sandwiched within the construction or on the inner faces of walls. Membranes are not normally applied to floor surfaces and left uncovered as they lack the necessary wearing qualities. If applied to tops of slabs, a protective slab (or similar) will need to be incorporated over the membrane, to hold it in place. There are a number of waterproofing systems available (see Chapter 5). Any chosen system should, as appropriate, be able to withstand the hydrostatic pressure and/or the effects of loading. Some waterproofing systems may also provide high resistance to vapour movement. In this type of construction the structure itself is not specifically designed to be watertight and effectively it may be designed to the requirements of BS EN 1992-1-1[9]: see Table 9.4.

Clearly, external membranes (or 'tanking') will only be suitable where there is access to the external face for initial construction. Scope for subsequent repairs will be limited by access, and locating the source of any defect in a system not continuously bonded will be problematic particularly as defects may only become apparent after construction. Internally applied membranes will be easier to maintain, but performance can be affected by hydrostatic pressures and attachments fixed through them post-construction.

It should be noted that external membranes prevent autogenous healing (see Section 11.4) of early-age cracks and encourage drying shrinkage cracks in concrete. In extremely aggressive ground conditions membranes may be used to protect the concrete structure.

Figure 3.2
Type A water-resisting construction (barrier protection)



The structure itself does not prevent water ingress. Protection is dependent on the total water barrier system or water and vapour barrier system applied internally or externally or sandwiched between structural elements in accordance with manufacturers' instructions. Edge thickenings are to be discouraged with external waterproofing.

Type B – structurally integral protection

Type B construction normally takes the form of a reinforced concrete box *without* reliance on applied membranes (see Figure 3.3). The box is designed to BS EN 1992-3[10] so that crack control minimises the risk of water penetration. Limits on crack widths depend on the water table and/or intended grade of use. Where the water table and risk are classified as being low, design to BS EN 1992-1-1 should be acceptable (See Table 9.4).

Without membranes, the structure is unlikely to be fully vapour resistant and additional measures may be required. Thus a type B basement may need to be converted to a type A or C construction. Alternatively and more usually, the effects of vapour penetration may be conveniently overcome by using heating and/or ventilation. Type B construction with the additional vapour barriers can achieve all grades of internal environment.

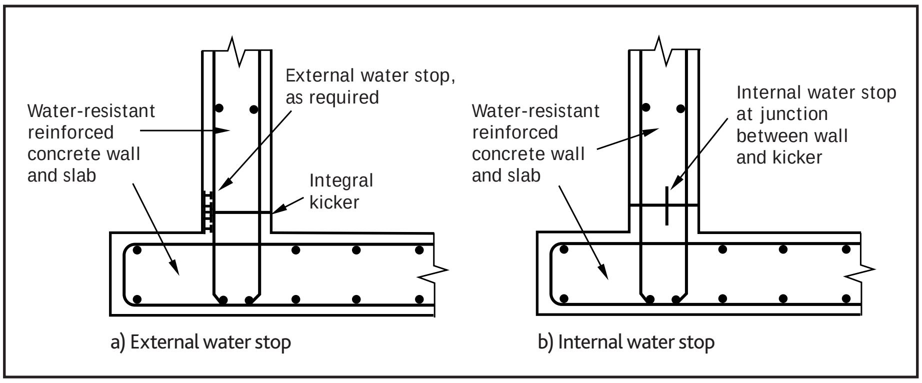
Good workmanship is essential to avoid defects that let water through. Common defects are permeable concrete caused by poor workmanship: poor compaction, honeycombed concrete, poor installation of water bars, and poor preparation and contamination of joints. Any water penetration through minor defects under high water table conditions can be repaired from the inside. See Section 11 for construction details.

According to BS 8102: 2009 [16], Type B construction may also be provided by piled walls: e.g. steel sheet, contiguous or secant piling or diaphragm walls. In high or variable water table classifications, piled walls must be:

- directly accessible for repair and maintenance from inside the structure, or
- combined with a fully bonded waterproofing membrane, or
- integrated with a facing concrete wall designed to achieve the desired crack control.

However, water penetration usually comes via the vertical joints between piles, therefore there is little benefit in designing them to BS EN 1992-3. As the requirements for the design of the piles is very dependent on temporary load cases for which the contractor is usually responsible, the design responsibility and the risks associated with design should be clearly addressed in the contract documentation.

Figure 3.3
***Type* B water-resisting construction (structurally integral protection).**



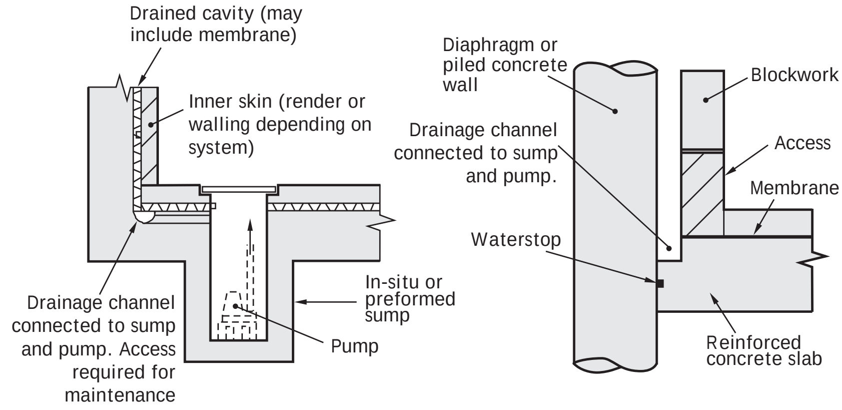
Type C – drained protection

Type C construction incorporates a drained cavity within the basement and relies on this cavity to collect any water from seepage and drain it to sumps for pumping out (see Figure 3.4). A dry internal environment can be achieved with confidence using a drained cavity wall and floor construction providing any defects are corrected and the system is maintained.

The external wall and base slab should substantially prevent ingress of water; otherwise the cavity and the pumps may not be able to cope with the flows. Large flows may also lead to loss of fines in the surrounding soils. Disposal of even small quantities of drained water may become an issue requiring negotiations with authorities. It should be noted that the water authorities and Environment Agency (EA) will generally not approve of large quantities of groundwater being pumped into sewers or rivers and special provision may be necessary to avoid loss of fine materials. If a drainage solution is adopted then maintenance requirements must be considered to cater for the drain or filter becoming blocked or defective. Ineffective drains and filters are likely to cause problems if maintenance allowance is not made.

As CIRIA Report R140[20] notes, the cavity should not be used to conceal large leaks. Where water bars are used, it is important that they are continuous covering all construction joints.

Figure 3.4
***Type* C water-resisting construction (drained protection).**



To avoid complications, sumps are often specified to be poured in one pour either with the basement slab using hanging formwork or by using central water bars in the top of the sump walls which are poured to the underside slab level.

a) Reinforced concrete wall with drained cavity to wall and slab

Provision of a ventilated cavity and horizontal damp-proof membrane minimises the risk of vapour ingress. Termination of membranes into cavities requires care to avoid 'damp bridges'.

b) Diaphragm or piled concrete wall with drained cavity to wall

Vapour transmission may be minimised by ventilating the cavity or providing barriers on the inside and providing an effective damp-proof membrane over the drained floor. There are many other possibilities. Where membranes are used precautions should be taken to prevent separation of any internally applied membrane from the substrate because of hydrostatic pressure. In the case of applied renders this may be achieved by providing mechanical anchorage. Adhesive sheet membranes may be hidden behind a non-structural inner skin.

Defects that can arise with this form of construction are flooding caused by failure of drains or pumps, or blockage of drains by silt or other sediments. Proprietary channels are sometimes incorporated at the base of the walls to collect the water ingress. Access should be available for clearing the silt and rodding the drains in the event of blockage. Access to the cavity behind some linings is not possible and clearly it will be advantageous to construct any inner wall or lining as late as possible so that any defects that appear may be seen and repaired.

3.3 Likely *forms* of reinforced concrete construction

The *types* of water-resisting construction manifest themselves in various forms of construction. Table 3.4 summarises the likely *forms* of reinforced concrete construction for different site conditions and uses of basement space.

Level of water table		Form of construction/ external wall	Method of construction	Water excluding property	Likely *grade* that can be achieved with different levels of vapour exclusion	
					Likely *grade*	**Additional measures**
Low ↑	**Generally below floor level**	RC box	In open excavation or within temporary works such as sheet piling or King post systems	Good if treated as Type B and designed to be water resistant (to BS EN 1992-3)	1 or 2	No additional measures
					3 (or (4))	External or internal membrane (Type A) or drained cavity (Type C) or active precautions[a]
				If not so designed, insufficient	Should be treated as Type A construction (relying on waterproofing membranes for water exclusion) or as Type C construction	
		Contiguous piling with subsequent concrete facing wall[c]	Basement excavated after piling with the floors acting as props in the final condition	Insufficient. Additional measures usually necessary	extnl. sunken areas only	No additional measures
					1 and 2	A designed concrete facing wall[c]
					1 and 2	A drained cavity and/or internal membrane[b]
Order of cost					3 and (4)	A drained cavity and internal membrane[b] and active precautions[a]
	Permanently above lowest basement floor level – variable to high	RC box	In open excavation subject to managing ground water	Good if treated as Type B and designed to be water resistant (to BS EN 1992-3)	1 or 2	No additional measures
					3 (or (4))	External or internal membrane or drained cavity and active precautions[a]
		Secant piling	Basement excavated after piling with the floors acting as props in the final condition	Usually insufficient. Fully bonded membrane, facing wall or drained cavity necessary[d].	1 and 2	A designed concrete facing wall[c]
					1 and 2	Drained cavity and internal tanking
					3 and 4	Drained cavity and/or internal membrane[b] and active precautions[a]
		Diaphragm walling	Basement excavated after piling with the floors acting as props in the final condition	Usually insufficient. Fully bonded membrane or drained cavity necessary[d].	1 and 2	A designed concrete facing wall[c]
					1 and 2	Drained cavity and/or internal membrane
↓ **High**					3 (or (4))	Drained cavity and internal membrane[b] and/or active precautions[a]

Notes

a Active precautions relate to heating and ventilation requirements to achieve the required internal environment.
b Fully bonded waterproofing membrane applied on the inside face of the structural walls.
c Facing walls may be designed to BS EN 1992-3, so where integrated with a designed slab form an RC box with the properties and likely grades indicated for RC boxes above.
d Piling/wall should remain accessible for repair and maintenance.

Table 3.4
Forms of reinforced concrete basement construction related to site conditions and use of basement space (based on CIRIA Report R140[20]).

3.4 Surveys and ground investigations

In order to achieve the most cost-effective solution, it is essential to gather as much information as possible about the site and surroundings.

The design team should advise the client on the need for exploratory works and have them commissioned as early as possible in the design process. Normally this starts with a desk study of the site initiated perhaps by commissioning a report from one of the environmental data agencies. Among other things it will cover the following: a review of the geological maps of the area; borehole records from the British Geological Survey; historic ordnance maps; a check on environmental matters such as contamination in the ground, water courses, hydrology and any relevant information obtainable through the Environment Agency; all relevant information from statutory bodies regarding under- and over-ground statutory utilities equipment. The findings of the desk study will determine the nature of detailed field exploration that will be necessary.

The detailed exploration should normally include the following:

- Subsoil investigation to determine ground conditions and groundwater levels for sufficient depth and in sufficient locations.
- Chemical tests of soil to establish any contaminants in the ground including the likelihood of gases. This information will be needed
  - for the design of the structure,
  - for observing precautions during construction, and
  - for the disposal of the contaminated soil.
- Site surveys to determine boundaries and relationship to adjoining buildings and roads. Railway lines and tunnels should be identified in the survey. If the site is adjacent to or over underground tunnels, close liaison with the relevant owners will be required to determine their requirements and ascertain any limitations that may be imposed.
- Location of incoming services and any services within and adjoining the site. Useful guidance on positioning of services in footways is given in The National Joint Utilities Groups series of guidelines[22].
- Determination of likely obstructions below ground (e.g. old foundations, cellars etc.) particularly where the site has been used in the past.
- Establishing foundation details of adjoining buildings likely to be affected by the proposed basement construction.
- Assessment of risk of flooding through liaison with the Environment Agency.

The objective of the subsoil investigation is to obtain geotechnical and environmental design parameters by testing representative samples from the site to enable design calculations (e.g. bearing capacity, design level of water table, data for pile design, effective stress analysis for determining earth pressures, settlements, sub-grade modulus etc.). Exploration may also be necessary outside of the basement plan area, for example if ground anchors are to be designed to provide support to the vertical walls.

The specification for the Ground Investigation should include for the determination of ACEC (aggressive chemical environment for concrete) Class and DC (design chemical) Class according to BRE Special Digest 1[23] and/or BS 8500-1[13]. The concrete producer should be advised of the DC Class.

Groundwater/ground conditions also need to be checked for chemicals which may have deleterious effects on internal finishes or waterproofing.

Recommendations of BS EN 1997-2[24] for ground investigation and testing should be followed. In particular the recommendations for exploration depth given in Appendix B3(5) and B3(10) of that document should be noted. As groundwater conditions are not usually known before exploration it is prudent to drill the first borehole to the maximum recommended depth (bearing in mind seasonal variations and the risk of penetrating an artesian aquifer). Subsequent boreholes may, or may not, be taken to such a depth. Note also that deeper boreholes may be required if piles are to be considered for the basement design.

Attention has to be given to the backfilling of boreholes taken within the footprint of the excavation such that the borehole does not provide a drainage channel for water in strata below the excavation base during the construction works. Grouting, using tremie pipe techniques, may be necessary.

BS 8102:2009[16] advises the designer also to take account of the presence of, or potential for, natural gases such as radon and methane when considering waterproofing. It should also be noted that high levels of radon can accumulate even where basements are protected by a waterproofing membrane. This may lead to the need to install a radon management system where the risk assessment (or tests in existing structures) indicates that legislation might otherwise apply. Further guidance is available[25] and designers should take note of the perceived risks from radon and advise their clients accordingly.

Although a thorough site investigation and ground investigation report might appear expensive, it will prove invaluable during design and construction and will pay the client dividends by minimising risks.

3.5 Precautions near underground tunnels, large sewers and service mains

If underground tunnels are present in the vicinity of the basement, measures should be taken so as not to endanger the strength and stability of the tunnels. The owner of the tunnel should be contacted early in the project to ascertain their requirements. Apart from underground railway tunnels (e.g. in London, London Underground (LUL)), there are also other underground tunnels (e.g. BT). Normally there is an approval procedure for the design. Organisations such as LUL will provide the designer with their standard requirements.

Typically these will include:

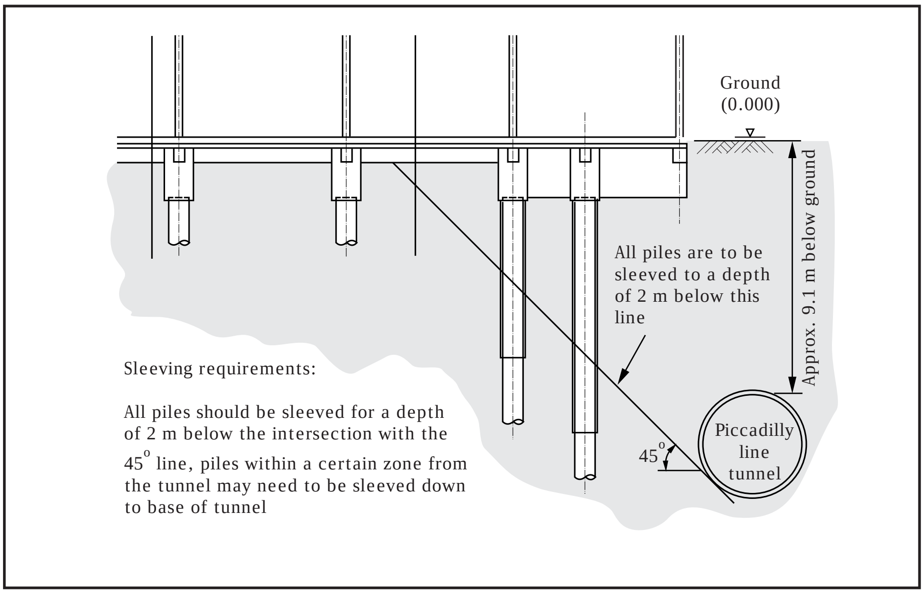
- A stipulation that no physical encroachment occurs within a prescribed exclusion zone around the tunnel.
- The need for sleeving of piles in the vicinity to prevent load transfer to the tunnels (see Figure 3.5).
- A maximum permitted horizontal and vertical surcharge on the tunnel.
- In sensitive cases, estimation of the expected distortions of the tunnel during construction and after the completion of the works.
- Surveys to establish the internal clearances in the tunnel, where they are already tight.
- Condition surveys and schedule of conditions before and after the construction.

Where deformations in existing tunnels are to be estimated, soil structure interaction will need to be considered. Normally, input by a geotechnical specialist will be required in such cases. Recourse to sophisticated computer programs will be needed, taking into account stress history, beginning with the construction of the tunnel.

The above requirements (e.g. exclusion zones) may influence construction methods such as piling type.

When large sewers or service mains exist close to the proposed basement, the relevant authority should be consulted to check their requirements.

Figure 3.5
Example of requirement for piles near tunnels.



3.6 Working adjacent to existing structures

For construction works in England and Wales, the Party Wall Act 1996[26, 27] sets out obligations of a developer, notably when excavations and/or piling are carried out adjacent to existing structures owned by others. If the development falls within the scope of the Act then the developer (through his advisors) has an obligation to serve notices on adjoining owners. Often, Party Wall surveyors will be appointed in order to prepare the Party Wall award to define the works, record condition of the properties, etc. There is an obligation for the developer to remedy any defects that manifest directly as a result of the works and usually to meet all reasonable fees and expenses of the adjoining owners' advisors. For excavations, the types of notices (3 m Notice and 6 m Notice) to be served are shown in Figure 3.6.

Normally condition surveys would be undertaken before and after the works to assess any damage caused by the works. It is also prudent to monitor any existing cracks or defects in adjacent structures by installing suitable tell tales, inclinometers or similar devices. Some Party Wall engineers may insist on soil structure interaction analyses be carried out to confirm that their structure will not be adversely affected by the new construction.

All structures that support the highway potentially require technical approval from the overseeing organisation for the highways. Procedures are laid out in BD2/05 of DRMB [60]. Where required, time will be necessary to gain acceptance of Approval in Principle (AIP), prior to commencement of work.

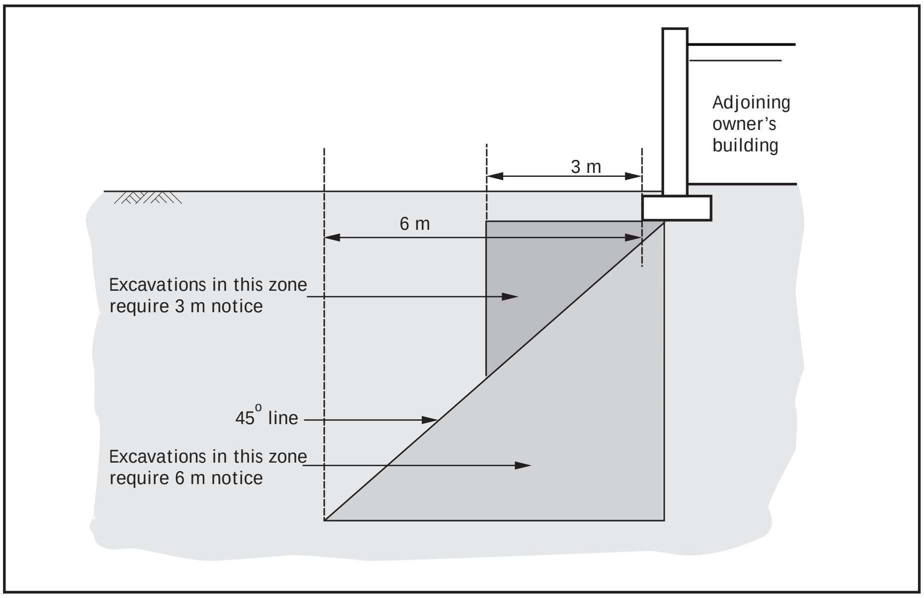


Figure 3.6
Statutory requirement for excavations adjacent to existing structures: rules governing requirement for 3 and/or 6 metre notices[27].

3.7 Tolerance of buildings to damage

If significant excavations are carried out adjacent to existing buildings, some soil movement is likely (see Section 4.1) and some damage to the existing building may result. Precautions should be taken to limit the degree of this damage. For example, the level of acceptable movement may determine the method of construction noting that cantilever walls will move more than walls that are propped or tied back. In the evaluation of risk, local knowledge or case histories may be consulted to estimate likely settlements. Tensile stresses cause most problems. For instance masonry walls in structures may be likened to brittle layered beams where hogging is critical.

Nevertheless, some objective criteria should be used to assess the damage. Damage can be categorised broadly as that which:

- affects appearance,
- affects the serviceability or functioning of the building or
- threatens the stability of the building.

The Classification developed by BRE[28] is widely used. This defines six categories of damage numbered 0 to 5 in increasing severity. Detailed discussions of the background can be found in Burland et al.[29, 30] and Skempton and MacDonald[31].

3.8 Space planning

While this will be carried out by the architect taking in to account the client's requirements, the structural engineer should be proactive and contribute to the process by explaining how the choice of retaining wall system will have an impact on the final basement space. The engineer should advise the architect early in the design process of the maximum physical internal dimensions of basement that are likely to be practical for the site. These are matters which can be reasonably easily identified by an experienced engineer prior to carrying out any design.

They include:

- the likely room for temporary works;
- clearances, especially any projecting features of adjoining structures and clearances for piling with respect to existing structures/boundaries (see Figure 3.7);
- restrictions imposed by owners of underground tunnels and utility companies;
- dimensions of guide walls for bored piles (may be around pile diameter + 800 mm);
- wall thickness;
- zone for cavity drains if relevant;
- tolerances for piling and temporary works (see Section 11.3.6).

And where using piled retaining walls:

- capping beams (allowing for temporary connections, temporary and permanent spans and load transfers; working space and services in adjacent ground; anchorage of pile reinforcement; tolerances; etc. (see Figure 3.8 and Section 11.3.7).

The net area of the basement can be significantly smaller than the gross site area and the architect and the client need to be realistic in their expectations.

The feasibility of using diaphragm walls should be raised with specialist contractors as their equipment takes up considerable space.

Figure 3.7
Typical site boundary clearances for bored piling.
(Courtesy of Cementation Foundations)

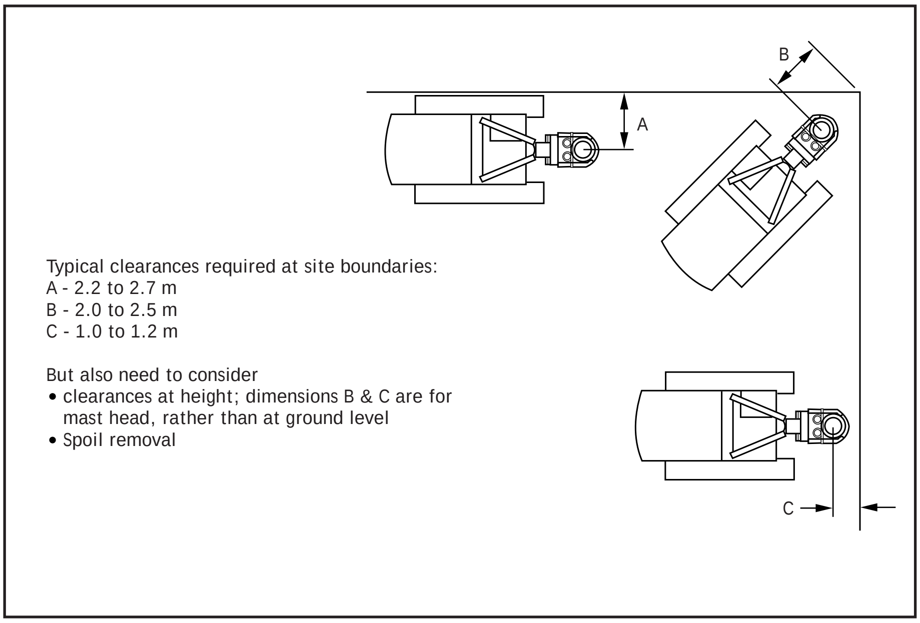
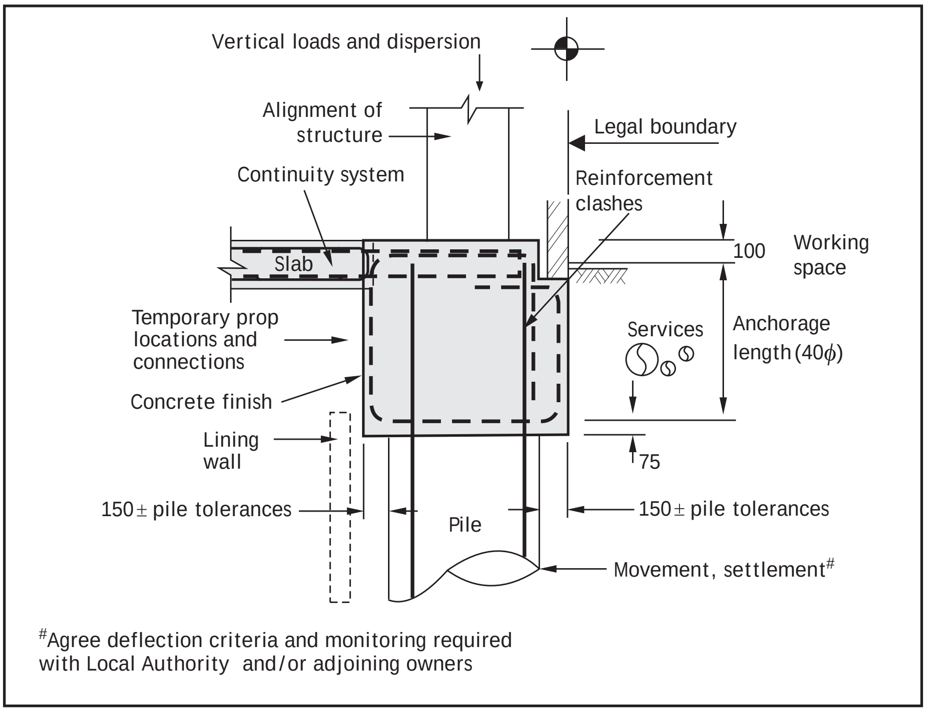


Figure 3.8
Capping beam issues.



3.9 Integrating the basement with the superstructure

The vertical elements of the superstructure such as columns, shear walls and lifts will need to continue to the foundations via the basement(s) unless transfer structures are introduced. Transfer structures are not uncommon to overcome the conflicting requirements above and within the basements. Clearly this will have cost implications.

Although there might be movement joints in the superstructure, generally they need not be reflected in the basement. It should be remembered that the environment in the basement is generally likely to be reasonably stable. Any double columns used in the external walls can normally be supported on external retaining walls without introducing a joint in the wall.

If the basement is to be used as a car park, it must be verified that the structural module of the superstructure, if continued into the basement, does not present difficulties for parking or traffic circulation, including sight lines. Further guidance is given in *Design recommendations for multi-storey and underground car parks*[32].

3.10 Fire safety considerations†

Requirements of the Building Regulations[5] should be met. Approved Document B[33] provides valuable guidance on the application of the regulations but is an extensive document with very detailed requirements. The size and depth of basement as well as proposed use and occupancy are important factors and there are special provisions for cases such as care homes, car parks and shopping complexes for example.

Strategic decisions relating to space planning and cost are briefly discussed below and should be made early in the design development. These include the need for alternative means of escape and firefighting shafts, use of sprinklers to reduce the fire resistance

† Although this section has been written citing the Building Regulations applicable to England and Wales, the principles will apply throughout the UK[5].

requirements, compartmentation and provision of ventilation. Designers should be aware of any structural implications that may result from the size of holes needed for vents and access. For example, a propped retaining wall may become and start acting as a cantilevered wall unless a waling can be incorporated at the top.

The following notes below are based on Volume 2 of the Approved Document.

Horizontal means of escape (*Section* 3): There should be a sufficient number of exits in each storey to allow safe escape for the occupants in the event of fire. The number of escape routes and exits to be provided depends on the number of occupants and the limits on travel distance to the nearest escape.

Vertical means of escape (*Section* 4): Sufficient number of adequately sized and protected stairs should be available. The number of escape stairs will be determined by:

- constraints on the design of horizontal escape routes;
- whether independent stairs are required in mixed occupancy;
- whether a single stair is acceptable;
- provision of adequate width for escape, while allowing for the possibility that a stair may have to be discounted because of fire or smoke.

Fire resistance (*Section* 7): All load-bearing elements must possess a minimum standard of fire resistance so as to prevent premature collapse or reduction in load-bearing capacity. Fire resistance is required to:

- minimise the risk to occupants;
- reduce the risk to firefighters;
- reduce the danger to people in the vicinity of the building. Minimum fire resistance periods are set out (in Appendix A) for different purpose groups, and in some cases are reduced if sprinklers are installed.

Compartmentation (*Section* 8): The spread of fire within a building may be restricted by sub-dividing it into compartments separated from one another by walls and/or floors of fire resisting construction. The object is to:

- prevent rapid fire spread, which could trap occupants;
- reduce the chance of fire becoming large, which is dangerous to the occupants, fire-fighters and people in the vicinity.

Firefighting access (*Section* 17): In buildings other than low-rise buildings (without deep basements) fire and rescue personnel need special facilities to reach the fire and work inside near the fire. This is to avoid delay and to provide a sufficiently secure base to allow effective action to be taken.

Buildings with a single basement at more than 10 m below the fire and rescue service vehicle access level should be provided with firefighting shafts containing a firefighting lift. Where there are two or more basements each exceeding 900 m^2 firefighting shafts should be provided but these need not include lifts. The number and location of fire-fighting shafts will be governed by maximum hose distances.

Venting of heat and smoke from basements (Section 18): The build up of smoke and heat as a result of a fire can seriously inhibit the ability of the fire and rescue service to carry out their tasks. Smoke outlets provide a route for heat and smoke to escape to the open air. They can also be used to let cooler air into the basements.

Performance of materials, products and structures (Appendix A): Table A2 of Approved Document B gives minimum periods of fire resistance for elements of structure. For the basements, these requirements are summarised in Table 3.5.

Table 3.5
Minimum periods of fire resistance for a basement storey and any floor over.

Purpose of building	Fire resistance required, minutes	
	Depth of lowest basement	
	>10 m	≤ 10 m
Residential	90	60
Office	90#	60
Shop, commercial	90#	60
Assembly, recreational	90#	60
Industrial	120#	90#
Storage and other	120#	90#
Car park (light vehicles)	90	90

Key
\# If the building is provided with automatic sprinklers throughout then periods may be reduced by 30 minutes.

3.11 Client approval

The design of basements requires consideration of a number of issues and as normal with any design a number of alternative solutions should be considered and budget costs prepared.

The costs should include the following: structure; temporary works; systems for achieving watertightness; systems for achieving vapour-tightness; services; finishes; provisional sums for repairing adjacent structures and highways.

A report should be prepared setting out:

- The client's requirements (the intended use and future use; the internal environment to be achieved; (Water) Tightness Class agreed for the structure).
- The forms of construction and options considered with an outline method statement of construction sequence and programme for each option.
- Budget cost for each option.
- Risk assessment for each option.
- The relative merits (e.g. costs, time, risks) of the different options stating the net area of basement achievable with each option.
- The recommended option together with reasons for the recommendation; and selected drawings to supplement the written report.
- A schedule of design responsibilities.

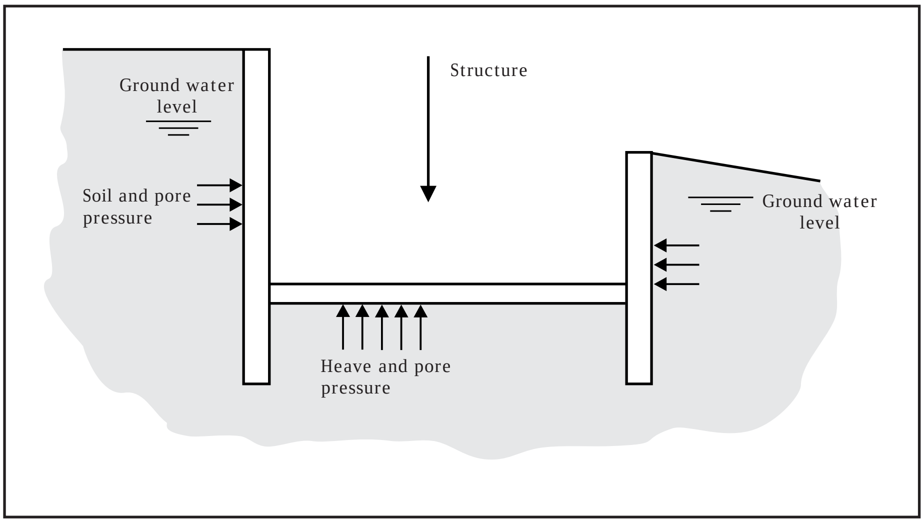
In view of the number of complex issues and interactions involved, it is important that the client's expectations should be managed and there should be a single point of responsibility[19]. The client should be asked to give written instructions to proceed to the detailed design stage on the basis of the report.

4. Ground movements and construction methods

4.1 Soil behaviour

Construction of basements involves excavation below ground level and is subject to actions illustrated in Figure 4.1. Excavations result in load relief in both horizontal and vertical directions. Therefore it should be appreciated that basements cannot be constructed without causing ground movements. These movements can be controlled by adopting appropriate construction techniques. Ground movement and earth pressures are highly dependent on the construction method and geotechnical conditions and precise predictions of these are difficult.

Figure 4.1
Actions on a basement.



Idealised behaviour of the mechanisms involved has been described by Burland[34], and is outlined here. It is important to distinguish the effects of vertical and horizontal stress relief that occur as a result of excavation. Excavation leads to horizontal load on temporary works. This induces deformation of the temporary works. As a result ground settles. Figure 4.2 shows the effect of horizontal load relief for:

a) a cantilever wall; and

b) a propped wall, notably the effect of deformations in temporary works on settlement.

In cantilever wall construction (Figure 4.2a) settlement at the surface has both vertical and horizontal components. In the case of a propped wall (Figure 4.2b) it is predominantly vertical. The timing and nature of the movements depend also upon the type of soil around and below the basement.

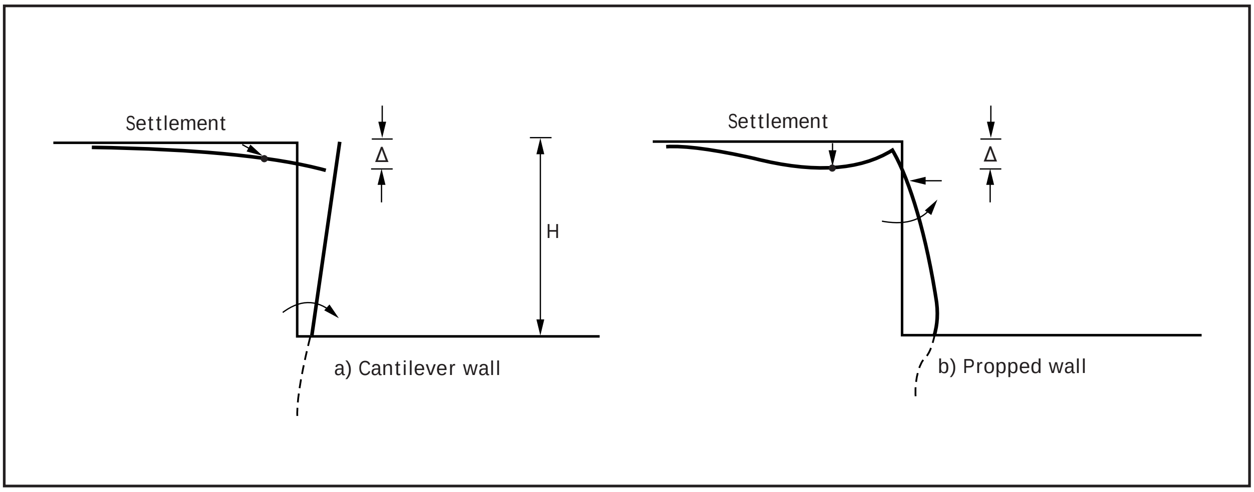


Figure 4.2
Effect of horizontal load relief on (a) a cantilever wall and (b) a propped (or tied-back) wall.

Figure 4.3 considers the effect of both downward and upward loads on a clay soil. Figure 4.3a shows the effect of downward load. In the short term, distortion of soil takes place at constant volume as no drainage of pore water pressure occurs (i.e. the undrained condition). As a result the ground outside the load tends to heave. In the long-term (when drainage occurs: the drained condition), settlement continues below the load and continues outside it. The upward load shown in Figure 4.3b may be thought of as vertical load relief. It can be seen that the reverse of the previous case occurs. Figure 4.3c idealises the base of an excavation. It is clear that even when the sides of the excavations are prevented from moving horizontally, settlements will occur in the surrounding ground in the short term and the surrounding ground will swell in the long term. Vertical stress relief at the base of an excavation can also induce deep-seated inward displacements that will not be controlled by props placed within the excavations.

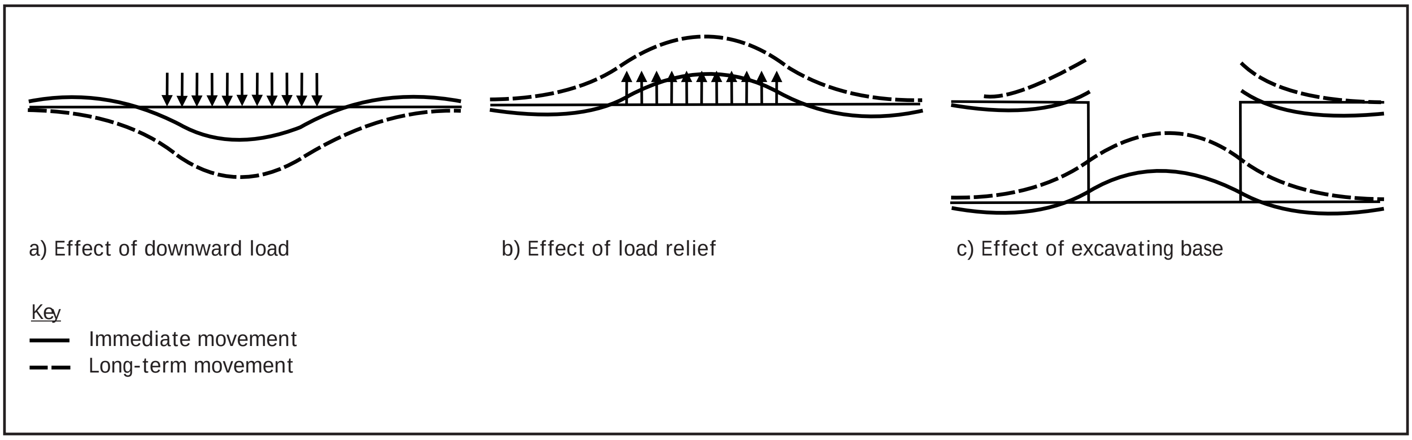
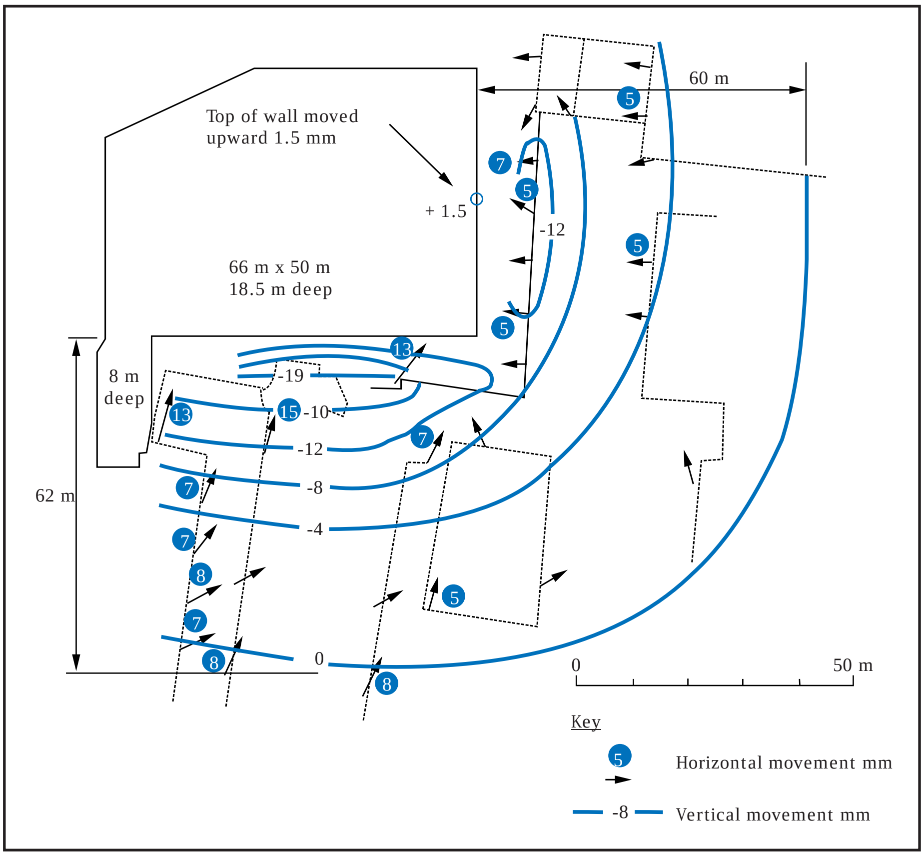


Figure 4.3
Effect of vertical load and load relief in clay soils.

Vertical movement might occur over a large radius around the excavation. For example during the construction of the 18.5 m deep car park at the House of Commons (top down construction within diaphragm walls), movement occurred up to a radius of three times the excavated depth although significant movements (about 5 mm or more) were within two times the depth (see Figure 4.4).



Right: **Figure 4.4**
Observed vertical and horizontal movements around the Palace of Westminster car park[35]

In high-permeability coarse soils the process of basement construction will result in an almost instantaneous response to changes in loads and groundwater conditions (i.e. fully drained conditions will develop quickly). The problems associated with granular soils are principally concerned with the control of groundwater to avoid the loss of ground due to high hydraulic gradients and movements during the installation of walls. Filtration is required in the design of the drainage system to prevent loss of fines. Should loss of fines occur, subsidence and ground movement are possible.

It is worth noting that the ground movement outside an excavation is influenced to a large degree by the rigidity of temporary works, the type of backfill, the rigidity of the basement wall itself and control of flow and filtration of groundwater. Utilisation of the permanent works to support the excavated face at all stages will help to minimise the movement. For embedded walls in stiff clay Carder[36] gives recommended upper bound values of horizontal and vertical movements at ground surface, see Table 4.1.

Table 4.1
Horizontal and vertical movements caused by excavation in front of embedded walls founded in stiff clay.

	Stiffness of support		
	High	**Medium**	**Low**
Examples of support	Top-down construction, temporary props prior to permanent props at high level	Temporary high stiffness props prior to permanent props at low level	Cantilever, temporary low stiffness props or props at low level
Upper bound values of horizontal movements	0.125% *h*	0.2% *h*	0.4% *h*
Upper bound values of vertical movements	0.1% *h*	0.1% *h*	0.2% *h*

Notes
1 *h* = maximum excavation depth in front of wall.
2 Maxima occur close to wall. Vertical movements are particularly variable.
3 Generally the extent of possible movements are up to four times the excavation depth.

Separately from these movements other movements can develop during construction. There is a risk of heave of the base of the excavation, and possibly failure, if permeable strata or layers containing high pore water pressures exist below the formation level (possible within a zone beneath the excavation down to about the depth of the excavation from ground level). Geotechnical advice should be sought when such conditions occur.

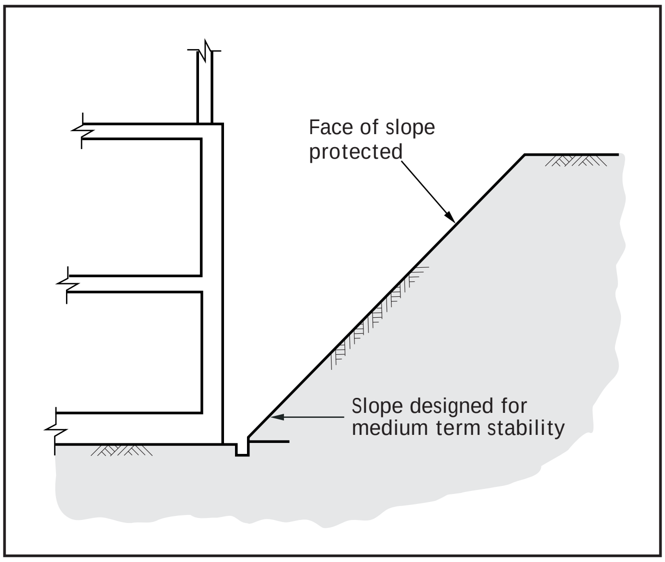
4.2 Methods of construction

From the previous discussion it is clear that the method employed to construct the basement plays a key role in controlling movements. Consideration of the construction method and sequence of construction is an integral part of every good design but critical in the case of basements. The designer of such structures should be fully aware of the different options available. Not all the methods will be suitable for every site. The different options relate primarily to the method of retaining the excavated face during construction and are discussed in *Design and construction of deep basements*[35] and elsewhere[37, 38]. A broad summary is provided here.

Open excavations

Where site conditions permit, ground can be banked (Figure 4.5). The face of the slope may be unprotected or protected against the adverse effects of weathering. A variant of this would be steeper slopes in conjunction with toe walls or nailed slopes.

Figure 4.5
Construction in open excavation.



Bottom-up excavation

This is a traditional alternative to open excavations. Temporary or permanent cantilever or propped retaining walls are constructed to allow access to construct the foundations and the basement from the bottom up, see Figure 4.6.

Top-down construction

This is particularly suitable for deep basements with a number of floors below ground. It provides scope for reducing overall construction time allowing simultaneous construction of superstructure and substructure. Permanent walls and floors are progressively constructed and used to retain the surrounding soil and groundwater. Generally it reduces the temporary works required. There are a number of examples and variants of this method, see Figures 4.7a and b. Also see *Design and construction of deep basements*[35] and *Risks in domestic basement construction*[37].

Semi top-down construction

In this method the ground slab is cast with large openings to permit easy removal of arisings from excavations below. The skeletal floor structure (cf. permanent walls and slabs) is designed to act as a frame to support the external walls.

Clearly a combination of the above methods will also be possible depending on the site conditions. Each requires temporary works, which will have effects on the programme and costs.

Groundwater

Any groundwater entering the excavations should be pumped out. Suitable filters or filter membranes may be required to minimise the risk of washing out fine soil particles. In sites with a permanent high water table, control of groundwater movement is critical but details of techniques such as dewatering and ground freezing are outside the scope of this guide.

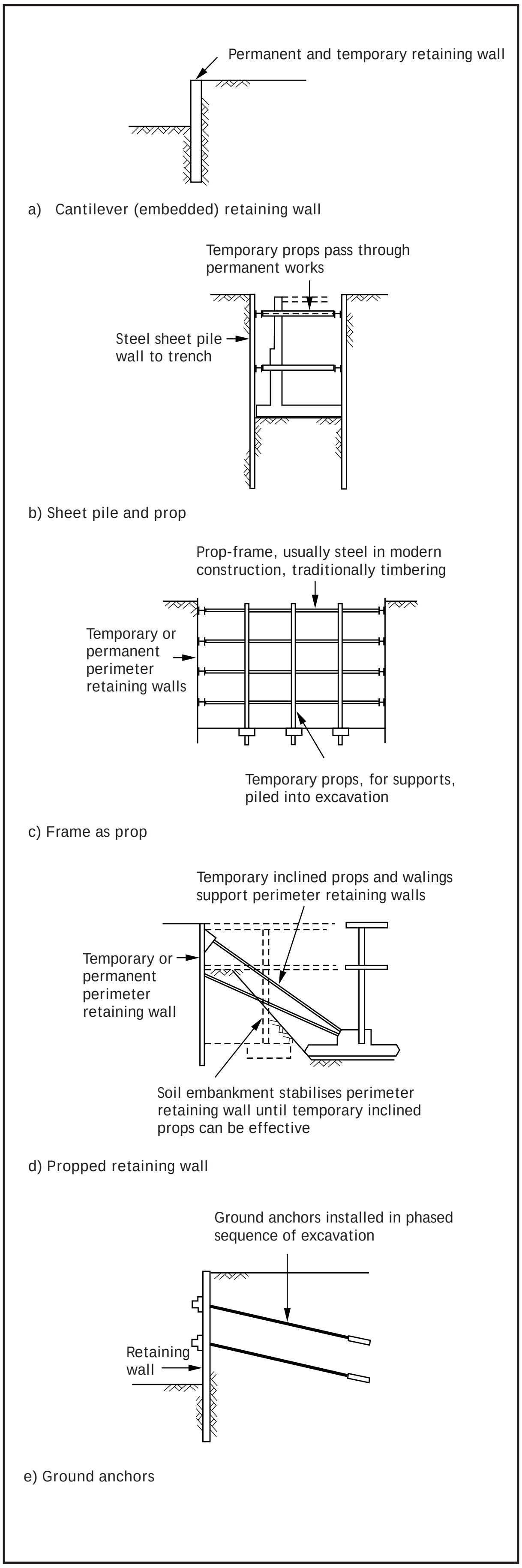
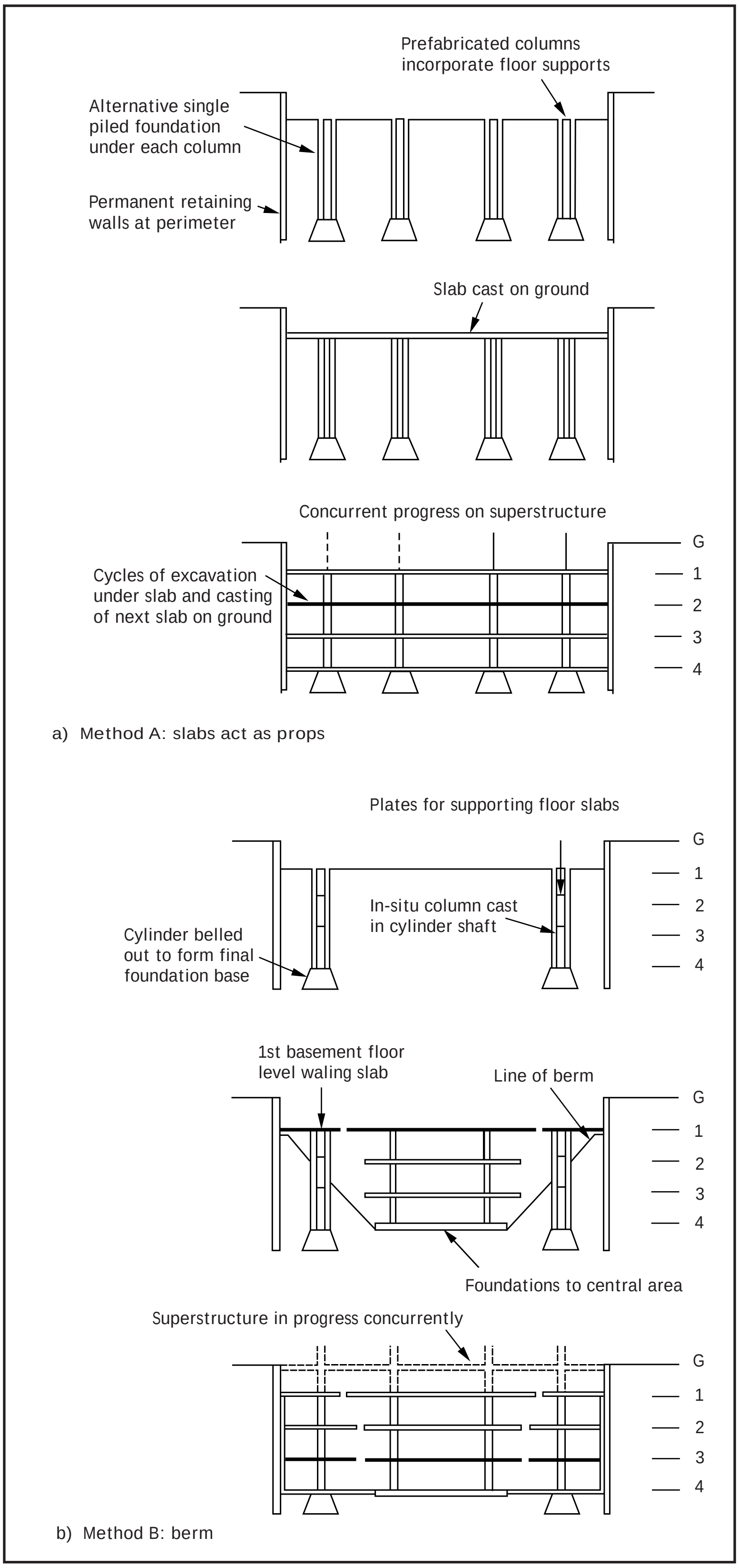


Figure 4.6 *Left.*
Bottom-up excavation (examples of alternative methods)[35].

Figure 4.7 *Below.*
Top-down construction (examples of alternative methods)[35].



4.3 Options for constructing basement walls

In open excavation

Where it is feasible to construct the walls in open dig it is usual to use in-situ concrete, although precast concrete and proprietary systems such as twin wall could be considered. In the case of precast and proprietary systems, specialist designers will become involved in the final design but the same design and construction criteria apply. In-situ and twin wall construction are illustrated in Figures 4.9 and 4.10.

Figure 4.9
In-situ basement wall construction.
Photo: John Doyle Construction

Figure 4.9 illustrates how in-situ walls were cast in open dig for a basement under a three storey office block on a sloping site in Sevenoaks. The soil was a silty/sandy clay and piled foundations were used. An external membrane was applied to the walls to help guard against water ingress. The columns support post-tensioned floors on a 9 m x 9 m grid. The basement is used for underground parking.

Figure 4.10
Twin wall construction.
Photo: John Doyle Construction

Figure 4.10 illustrates twin wall, which comprises two precast concrete slabs connected by means of cast-in lattice girders to form a single unit into which concrete is poured on site. The precast 'biscuits' are reinforced and the in-situ filling is reinforced as required, principally to provide continuity. It may also include insulation. The advantage is speed. The 72 m length of twin wall used as a basement wall was constructed by three men in three days. At joints

structural continuity is provided by the in-situ section alone. At construction joints proprietary water bars are used. These consist of coated galvanised steel plate 1.2 mm thick embedded up to 50 mm into the concrete and secured using proprietary clips.

Incorporating temporary retaining walls

A number of alternatives are available where temporary external retaining walls become part of the final structure. Table 4.2 lists the different retaining wall types and their relative merits. These include (see Figure 4.11 and 4.12):

- a King post (also referred to as panel and post) system
- sheet piles,
- contiguous bored piles,
- secant piles
- diaphragm walls.

For further details of, and guidance on, vertical embedded retaining walls, readers are directed to CIRIA C580 *Embedded retaining walls*[39], which provides best practice guidance on their selection and design. It covers temporary and permanent retaining walls that are supported by embedment in stiff clay and other competent soils.

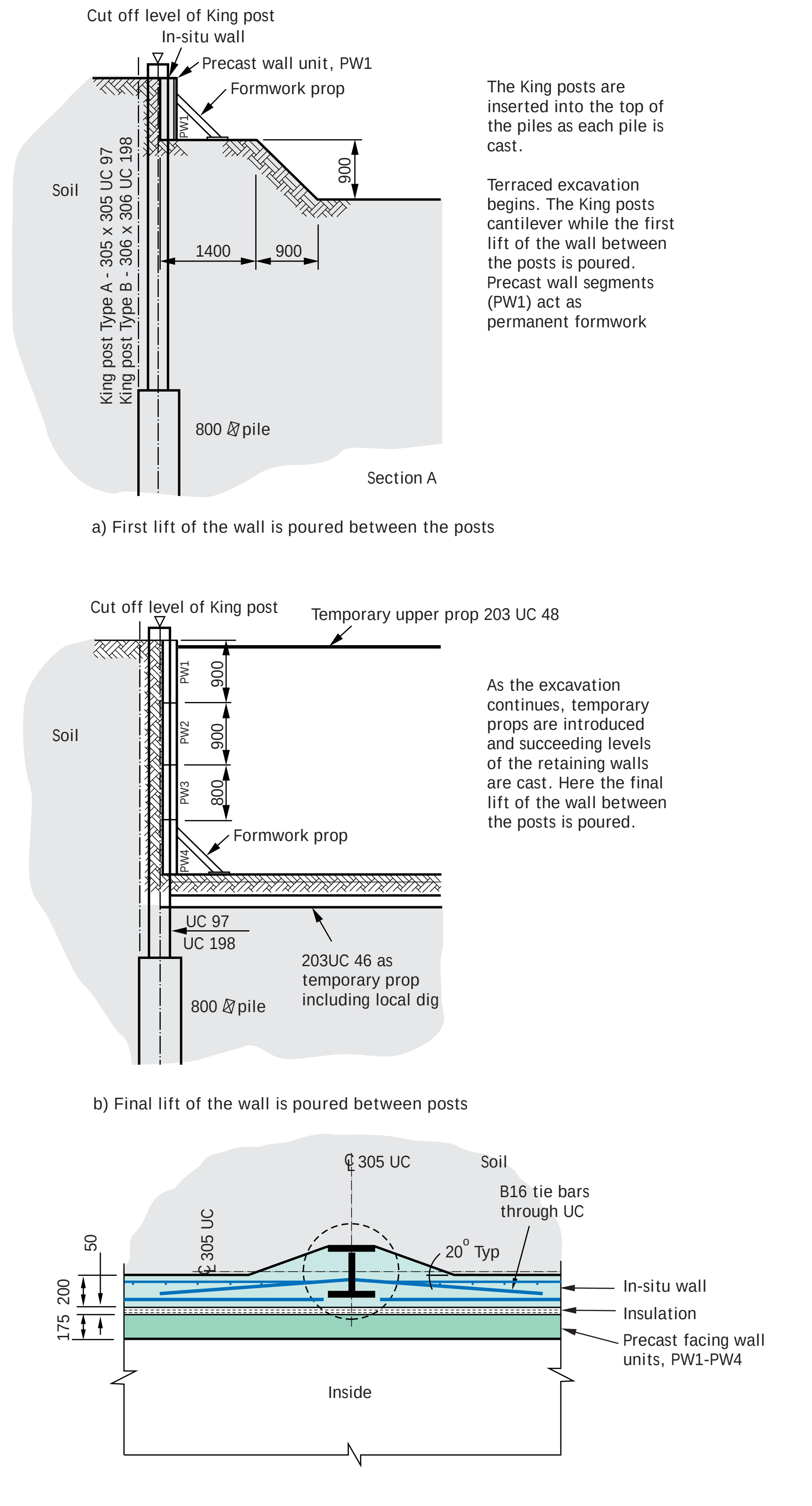
A typical detail for connection between a piled wall and ground slab is given in Figure 4.13.

Table 4.2
Embedded wall types used for soil support in basement construction$

Embedded wall types	Typical depth of piles/walls	Typical height of retention	Advantages/disadvantages	Remarks
King post wall: timber or concrete planks spanning between steel or timber soldiers at intervals	6–20 m	3.5 m as cantilevers or 12–15 m as propped or anchored posts	Not feasible in soft or loose soils nor with groundwater above the formation level	Walls are temporary and could act as permanent back shutter for RC walls
Steel sheet piling	10–15 m	8–12 m as propped wall	Vibration and noise could be a problem. In some types of soils they could be overcome by the use of hydraulic press equipment as used in the silent piling techniques	Walls could be either temporary or permanent. Reuse of sheet piles may determine the economic viability
Contiguous RC piles	12–20 m	6–15 m as propped or anchored	Will not produce a watertight wall	Walls act as permanent retention. To exclude soil and water, jet grouting could be used or the ground may be cut out between piles and backfilled with concrete or no-fines concrete
Secant piles – Hard/soft and hard/firm. Female piles use weaker concrete (or cement/ bentonite mix)	12–20 m propped or anchored	6–15 m	Weak concrete in female piles could pose long-term durability problems	Walls act as permanent retention. May be considered water resistant only in the short term. Pile tolerances may become an issue at depth
Secant piles – Hard/hard Female piles use normal concrete	15–30 m	10–20 m propped or anchored	Depth limited by vertical tolerances which influence the depth of cut joint and their water resistance. A minimum overlap of 25 mm is required at depth	Walls act as permanent retention. Female piles may be reinforced with UB sections. Male piles may use rebar cage or UBs. Pile tolerances may become an issue at depth
Diaphragm walls installed by grab	15–30 m	12–25 m propped or anchored	Heavy plant and difficulties in disposal of slurry are disadvantages	Walls act as permanent retention. Good solution for deep walls in variable soils with water retention
Diaphragm walls installed by cutter	15–50 m	12–35 m propped or anchored wall	Large mobilisation and demobilisation costs. Will be suitable only for large jobs	

Note
$ adapted from Table 4.1 in *Design and construction of deep basements*[35]

Figure 4.11
Example of excavation using King-post construction.

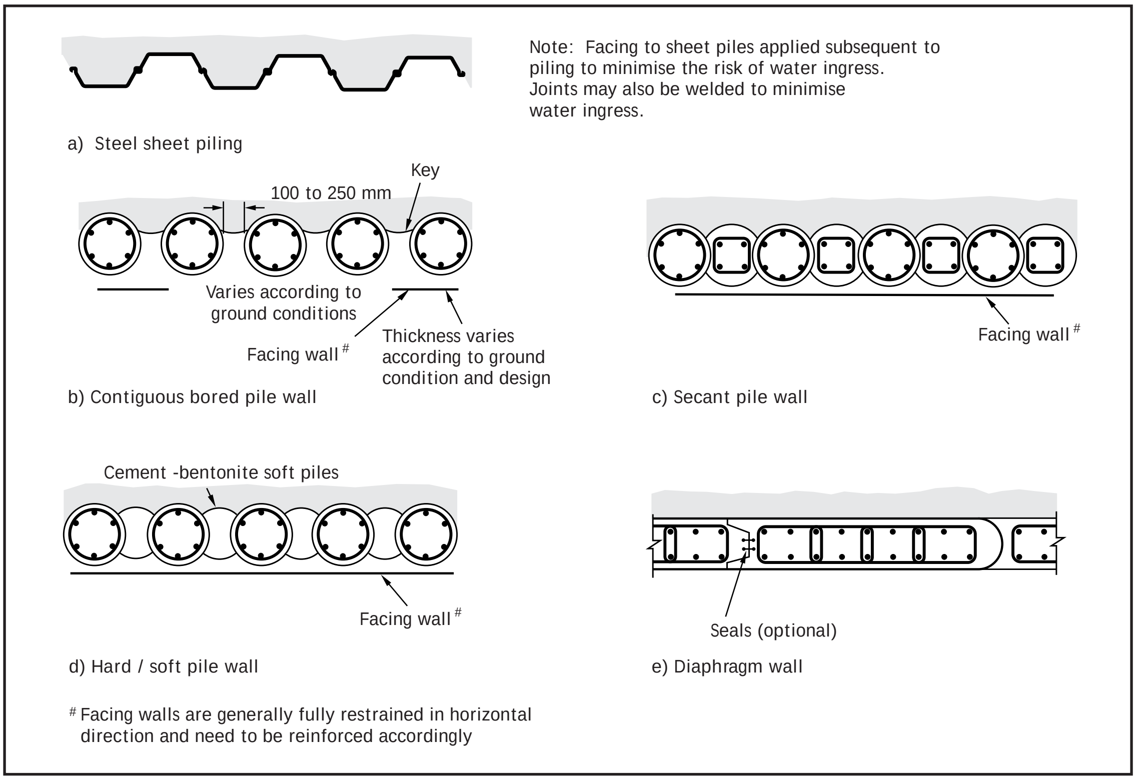


a) First lift of the wall is poured between the posts

b) Final lift of the wall is poured between posts

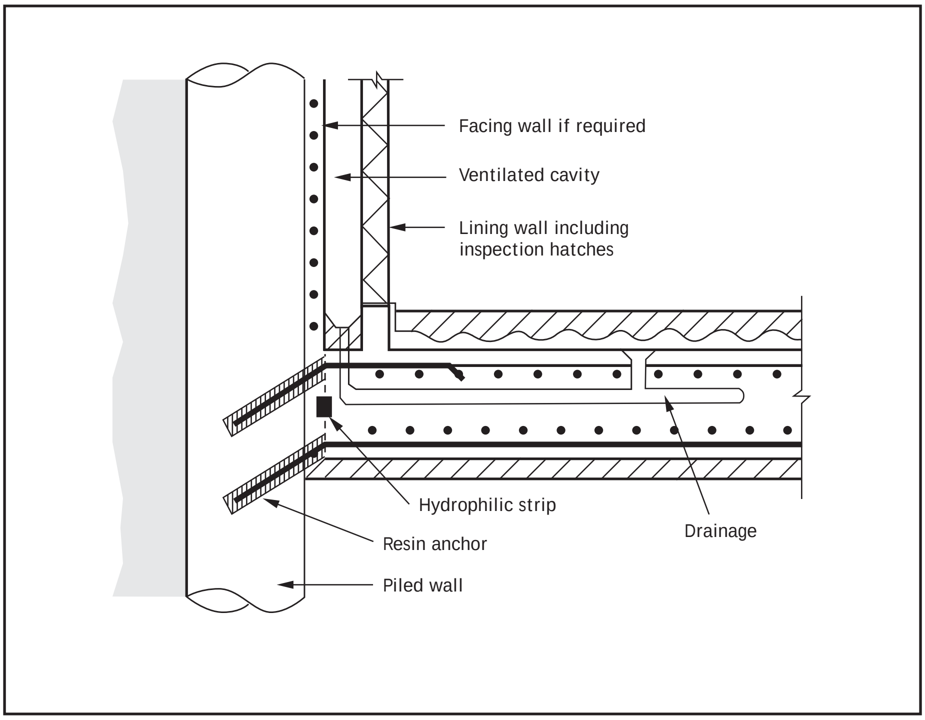
c) Plan section showing the King post and the wall between posts

RC details are indicated.
The backing wall can be in timber, precast or in-situ concrete, or steel sheeting.



Above: **Figure 4.12**
Options for embedded retaining walls

Right: **Figure 4.13**
Connection between a piled wall and basement slab



Facing walls

Non-structural facing (or lining) walls are used in theoretically dry ground conditions to provide nominal protection to the soil, improve aesthetics and/or enhance (but not necessarily assure) the water resistance of a piled wall system (see Figure 4.14). For general stability and robustness, they may be tied back to the piles.

In variable or high water tables, BS 8102[16] Type B construction requires facing walls to be designed to Eurocode 2. They should therefore be designed for lateral earth and ground water pressures and for the effects of restraint. If the walls are cast directly against the undulating profile of piles, considerable amounts of reinforcement will normally be required to control crack widths. It may be more prudent to provide a separate permanent box inside liner walls. In addition to the reasons given above these liner walls are cast to minimise in-plane restraint to the inner walls and to take out variations in pile alignment.

Sprayed concrete might be a suitable option. It may be used to fill gaps between piles, fill gaps and provide a 50 mm thickness for a more uniform contoured appearance or with additional thickness can produce flat structural walls, reinforced as required.

Figure 4.14 illustrates a basement constructed adjacent to a canal in Islington. The piled wall has a reinforced concrete facing wall, which was poured full height to give a full height fair-faced finish. The one-sided formwork to the walls meant that there were no through bolts. Falsework frames were used in preference to raking props but the frames required large fixing anchors (left foreground) which themselves needed a minimum of 300 mm depth in the slab concrete.

The watertightness of the slab was augmented by use of a pre-hydrated bentonite waterproof membrane.

Figure 4.14
RC facing wall being constructed.
Photo: GCL

4.4 Temporary works

The contractor is normally responsible for the detailed design of temporary works. Nevertheless, as has been emphasised, the method of construction and the temporary works required to achieve it will generally influence the design of permanent structures. The designer should thus be familiar with temporary works. A method statement indicating the construction sequence assumed in design should be prepared at tender stage. Detailed treatment of temporary works is outside the scope of this guide and the

reader should refer to other publications e.g. *Deep excavations: A practical manual* by Malcolm Puller[40]. Temporary props will sometimes pass through the permanent walls. It may be possible to remove the props progressively as sections of wall are built. If such sequencing is either not feasible or not considered economic, the props should be treated as penetrations through walls and suitable precautions should be taken to minimise the risk of water penetration (see Section 11.3.5).

Figure 4.15 shows a 750 mm diameter contiguous piled wall used as temporary support for works where the formation was 9 m below road level and 7 m depth below the adjacent retained facade's foundation level. Steel walings were used in lieu of a capping beam. The piles were later lined with shotcrete and a waterproof membrane before construction of a reinforced concrete wall.

Figure 4.15
Temporary works at Lancaster Gate.
Photo: Wentworth House Partnership

Figure 4.16 illustrates how basement construction can lead to 'busy' sites with programme implications. Note how the props pass through the facing wall and the basement space quickly becomes congested.

Figure 4.16
Basement construction
Photo: GCL

5. Selection of materials

Good concrete is inherently water resistant. Before considering selection of materials, it is worth emphasising the fundamental requirements for achieving such performance.

These are:
- correct structural design;
- appropriate concrete mix;
- good workmanship in construction; and
- appropriate supervision.

Many water-resistant basements have been successfully constructed on the basis of the above alone. There are a number of admixtures in the market to modify the properties of the fresh and hardened concrete (e.g. porosity, permeability). Such measures should not be necessary if the basic principles noted above are observed. Designers, who are inclined to specify such admixtures, should critically evaluate them and their cost benefit, bearing in mind that the basic requirements cannot be relaxed as a result. If reliance is placed on warranties for the products, the client should be apprised of limitations and implications.

Concrete is *not* vapour tight. A vapour-tight environment can only be achieved by incorporation of membranes, adequate heating and ventilation.

This section discusses the different types of materials commonly considered.

5.1 Concrete

Concrete composition should be carefully selected to achieve water-resistant construction bearing in mind the method of placing. Concrete should be specified in accordance with BS 8500[13]. In basement structures a critical design consideration is crack control, with strength generally not a critical criterion. Therefore, mix design should aim at durability and minimising the risk of cracking. Water resistance and durability can be achieved using good quality concrete alone.

Minimum reinforcement to control cracking and reinforcement to control the width of cracking due to early thermal and/or long-term drying shrinkage are both directly proportional to the tensile strength of concrete, which itself is proportional to the compressive strength. The risk of early age thermal cracking increases with cement content and increasing section thickness, but the use of cements incorporating ggbs, fly ash or limestone fines is beneficial in reducing the temperature rise[18]. Drying shrinkage is related to the initial free water/cement ratio of the concrete. Increasing the cementitious content should reduce the risk of drying shrinkage but due to the risk of thermal cracking it is preferable to reduce the amount of water using a water reducing admixture. A balance has to be struck between strength and durability requirements. With respect to finish, a dense closed particle surface is considered desirable.

With respect to BS 8500 the selection of concrete should follow the path of considering exposure Classification, and choosing appropriate strength and cover[13, 41]. The composition of the concrete should be determined by taking into account the requirements for the exposure Class on the soil and internal faces separately. It is likely that a *Designed concrete* will need to be specified in non-benign soils. *Designated concretes* are unlikely to be appropriate because FND designations have a strength Class of C25/30, other designations are considered unsuitable in foundations in Class AC-2 (or more highly aggressive) soils and no designation is considered suitable for use in chloride exposure Classes XD or XS.

For concrete in contact with soil, the ACEC (aggressive chemical environment of concrete) Class and DC (design chemical) Class should have been determined in the Ground Investigation in accordance with BRE Special Digest 1[23]. For concrete not in contact with soil, the exposure Class should first be determined in order to select the appropriate composition for concrete.

Benign soils

For basement walls and slabs where the ground conditions are benign (AC-1 ≡ DC-1 ground conditions), exposure Class XC3/XC4 (carbonation attack: moderate humidity or cyclic wet and dry) with associated XF1 (freeze/thaw attack: moderate without de-icing salts) is generally deemed appropriate. For minimum cover, a C30/37 concrete may be recommended. The requirements for a RC30/37 in BS 8500 are similar to the requirements of traditional specifications that have proved successful in the past, viz:

Table 5.1
Requirements for a RC30/37 designated concrete.

Requirement	Value
Maximum water/cement ratio	0.55
Minimum cementitious binder content	300 kg/m^3 when $h_{agg} \leq 20$ mm 320 kg/m^3 when $h_{agg} \leq 14$ mm
Consistence Class (Recommended)	S3

Cement and combination types should be limited to IIB-V (CEM I + 21% to 35% fly ash) and IIIA (CEM I + 36% to 65% ggbs) and specified accordingly. These cements will fall into cement type N of BS EN 1992-1-1 (see note to Table A.5)

Aggressive soils

When in contact with aggressive soil (AC-2 to AC-5 i.e. DC-2 to DC-4 ground conditions), provisions to resist the sulfate attack are likely to control the mix. In these cases the concrete producer should be advised of the DC Class. For the most aggressive soils, cement and combination types IIIB (CEM I + 66% to 80% ggbs) or IVB-V (36% to 55% fly ash) may be specified. These will correspond to cement type S of BS EN 1992-1-1. When considered suitable for DC-2 conditions, the appropriate Designated concrete is FND 2 whose the minimum strength Class is C25/30.

Car parks

Where the basement may be used for car parking, durability requirements should be determined from BS 8500[13]. However this standard does not address car parks

specifically and some interpretation is required. The risk of traffic bringing chlorides into the car park must be assessed but soffits, columns and walls are rarely exposed to spray that includes de-icing salts.

Generally a concrete Class of at least C32/40 is recommended provided:

- de-icing salts will not be applied directly to the elements as part of a maintenance regime.
- the car park will be well-drained.
- the car park will have good ventilation.
- the car park is located in the UK.
- 50 year design life.
- freezing of internal elements is unlikely to occur.
- elements are not immediately adjacent to a highway.

In other conditions reference should be made to BS 8500[13]. If abrasion is considered to be an issue (e.g. at the entry level), BS EN 1992-1-1 Cl. 4.4.1.2(13) advises that a sacrificial layer of 5 mm of concrete may be used.

Car parks protected with waterproofing may have reduced exposure, conditions, but consideration should be given to the maintenance regime.

Reducing the risk of cracking

As Section 9.10 points out measures to reduce the risk of early thermal cracking include possible specification of cement replacements, superplasticisers, water reducing agents and/or aggregates with high strain capacities and low coefficients of thermal expansion.

5.2 Waterproofing membranes and systems

Materials used for waterproofing should be appropriate for the proposed location, conditions and any anticipated movements. There are many proprietary systems in the market. It will be prudent to choose systems with Agrèment certificates, but the designer should consider the implications of any limitations cited in the certificates. Special care should be taken where a vapour-proof system is required. In general systems should not be mixed.

There are a wide range of products used for structural waterproofing. For ease of understanding, they have been separated into seven distinct categories[43] according to product type, form and application. Apart from Category 2, they are regarded as barrier systems for use as Type A protection (but may be combined with Type B protection). Category 2 creates a drainable cavity and is part of a Type C protection system. A brief summary of each category is provided below.

Category 1 – Bonded sheet membranes

These are cold-applied or heat-bonded to the structure. They are flexible and can accommodate minor movements. There are also composite sheet membranes, which can be fixed to vertical formwork or laid on the ground prior to pouring the slabs.

Category 2 – Cavity drain membranes

These are high-density polyethylene sheets placed against the structure. The dimples form the permanent cavity. These are generally used internally. They are flexible and are able to adapt to minor settlement and shrinkage of substrate. These are not waterproofing membranes in themselves; but facilitate drainage of any water ingress (see Figure 5.1).

Category 3 – Bentonite clay active membranes

These are sheets of sodium bentonite clay sandwiched between two layers of geotextile or biodegradable cardboard. When the clay meets water, it can swell to many times its original volume sealing any gaps or voids in the membrane. This category of membrane is used externally. Bentonite systems can be either bonded or unbonded. Where bonded, the system is simple to apply with minimum preparation of the substrate. The efficacy of the system under alternating drying and wetting conditions should be verified with manufacturers. It should not be used in acidic or excessively alkaline soil.

Category 4 – Liquid applied membranes

These one- or two-part systems are applied cold as a bitumen solution, elastomeric urethane or modified epoxy. If applied internally, a loading coat (a layer of material designed to hold a Type A waterproofing material in place when resisting water pressure), sufficiently strong to bond to a suitable substrate and withstand hydrostatic pressure, will be required. They can be used just as a vapour barrier in Type B protection where the structure will withstand the loading. Being jointless, the continuity of the membrane is maintained. It is easily applied although good surface preparation is necessary. When applied externally, they can protect the structure against aggressive soils and groundwater. Being elastic and flexible, minor movements in the substrate can be accommodated.

Category 5 – Mastic asphalt membranes

These are applied in three coats as a hot mastic liquid. They cool to a hard waterproof coating. The application may be external or internal but will need a loading coat if applied internally. Risk of defects in the coat being carried through all the coats of the membrane is low. Externally applied membranes are generally unsuitable for complicated foundation profiles.

Category 6 – Cementitious crystallisation active systems

These slurry coatings react with free lime in concrete, renders or mortars and block hairline cracks and capillaries. The chemicals remain active and will self-seal leaks. These products will not waterproof defective concrete (e.g. honeycombed areas).

Category 7 – Proprietary cementitious multi-coat renders, toppings and coatings

These coatings usually incorporate a waterproofing component and are applied in layers generally internally but may also be external. They are effective against severe ground water infiltration. Mechanical fixings through the system should be avoided.

Figure 5.1
Cavity drain membrane.
This cavity wall membrane has been fixed back to the retaining wall.
Photo: Waterman

Admixtures for watertightness

In addition, a number of proprietary admixture products are available. The essential components used will generally comprise a water-reducing superplasticiser, a hydrophobic component and a pore-blocking ingredient. The concrete mix will need to be designed taking into account the properties of the admixture(s). Providing the admixture has been assessed and certified and the design of the concrete mix and casting is adequately supervised, such concretes should provide increased resistance to water and water vapour (see BS 8102[16]). While a dense matrix of concrete will result in the body of the concrete, joints in the construction will remain vulnerable for water ingress and they will need to be protected by other means.

5.3 Water stops

Whilst opinion is still divided on the need for water stops (also referred to as water bars), additional protection at vulnerable locations (i.e. construction joints) is considered very desirable.

There are three broad types of pre-construction water stops and various post-construction techniques.

5.3.1 Preformed strips

Preformed strips of durable impermeable material are wholly or partially embedded in the concrete during construction. They are located across joints to provide a permanent watertight seal during the whole range of movements. Water bars may be proprietary products made of rubber or flexible plastics such as PVC. Where movement has to be accommodated these water bars incorporate a central bulb. The separate lengths of water bars should be heat fused to form a continuous barrier as illustrated in Figure 5.2. Section 11 shows typical details of joints incorporating metal and PVC water bars.

PVC water bars can be placed either within the concrete or on the external face. The latter are sometimes referred to as rear guard water bars and are only suitable when

there is external pressure to retain them in place (e.g. ground pressure). External water bars should not be used with external membranes. PVC water bars installed within the concrete need to be securely fixed to retain them in place during concreting. This is often ignored on site causing the water bars to collapse during concreting.

Figure 5.2
PVC water bar being 'welded' on site.
Photo: Waterman

As an alternative to PVC water bars at construction joints (both horizontal and vertical), a rigid water bar formed from a strip of black steel typically 1.5 mm thick (unpainted and non-galvanized) has proved effective (See section C3 of BS 8007[42]). Their stiffness ensures that they will not collapse during concreting and the water bar is placed centrally across the joint. In horizontal joints the water bar is gently pushed in when the concrete is still green, hence timing of this operation is quite critical. Separate lengths of the metal water bars need not be welded together. At butt joints a gap should be left equal to aggregate size + 5 mm, with a cover strip overlapping the two water bars, again leaving a gap of aggregate size + 5 mm (see Figure 11.4b). A similar detail should be used at corners.

5.3.2 Water swellable water stops

These depend upon a sealing pressure being developed by the water absorption of a hydrophilic material or filler. There are a number of proprietary products and they are available as strips for bonding or nailing to the concrete of the first pour immediately before making the second pour (see Figure 5.3). Their use is limited to joints where the movement is low. The effectiveness of the system under conditions of alternating wetting and drying should be verified with the manufacturer.

5.3.3 Cementitious crystalline water stops

These differ from the previous two types in that the product consists of cements, fillers and chemicals to be mixed on site as a slurry, which is to be applied to the face of the concrete of the first pour before the second pour. The water-stopping action results from salt crystallisation, in the presence of water, within the pores and capillaries of the concrete. These products are not suitable for use in expansion joints.

Figure 5.3
Hydrophilic water stop in position on top of an integral kicker.
Photo: Waterman

5.3.4 Miscellaneous post-construction techniques

These techniques include provisions for dealing with joints.

Injectable water bars

At construction joints, perforated or permeable hoses or tubes or resin injection channels are fixed to the first pour of concrete. Either end of the hose is attached to fittings that are connected to the formwork or protrude from underneath. The fittings are then cast into the joint and after temperature and shrinkage movements have stabilised sufficiently, they are used to inject a resin or other proprietary fluid under pressure: the resin flows out of the hose or channel into cracks, fissures or holes in the joint. The injected material then sets to seal all the water paths through the joint. With some systems re-injection is possible.

Figure 5.4
Injection hose system placed in a horizontal joint.
Photo: Max Frank Ltd

Rebate and sealant

Another relatively simple technique is to construct the joint with a water bar in the usual manner but leaving a small rebate (about 20 mm wide × 15 mm deep) on the accessible face of the joint. If water seepage occurs at a future date after construction, the rebate can be filled with a suitable sealant or grout. It is implicitly assumed that water penetration will occur at the joints and not through the body of the concrete. This should be the case if the structure is constructed correctly using an appropriate concrete mix and adequate reinforcement.

6. Structural design – general

This section covers structural options that are available for the basement slab and external walls together with the loads and other factors that should be considered in design. Design concepts should be integrated with the construction method used for the excavation of the basement and the needs for buildability and repairability.

6.1 Structural options for basement slabs

Where the loads from the superstructure are supported by separate foundations (e.g. pad/strip foundations or piles), a continuous basement slab is cast over the foundations. If the soil stratum at the level of the basement slab is able to support the weight of the slab and loads applied to it, the basement slab could be designed as ground bearing for downward loads. However, the effects of upward loads from water pressure and heave should be considered.

Where the upward forces due to heave or buoyancy are relatively small, the basement slab should be designed to withstand them by spanning between foundations capable of resisting the net uplift forces. The foundations for superstructure columns will be ideal as the dead loads from the superstructure can be utilised to counteract the uplift forces. In some cases tension piles may be necessary.

Where the uplift forces are significant, it may be prudent to leave a void under the slab to prevent heave acting on the basement slab. There are proprietary void formers in the market to create the void.

Upward buoyancy forces caused by groundwater should generally be resisted by the structure. Effective drainage is the only way to prevent buoyancy forces developing. However, where effective drainage is not allowed or is prohibitively expensive, allowing the basement to flood may be a strategy to be considered.

Where the strength of the soil stratum at the level of the basement slab is adequate to support the weight of the slab and loads directly applied to it, including loads from superstructure, raft slabs may be considered. The downward loads, some of which will only be applied at discrete locations, will cause upward earth pressure on the underside of the slab.

A variant on the above is to incorporate piles in the raft. This will help reduce settlements. The thickness of the raft may also be reduced as a result.

Structural analysis of basement slabs

Structural analysis of rafts is a complex problem as it involves soil structure interaction. Simplifying assumptions such as uniform pressure distribution can be unconservative or economically wasteful.

In the past, engineers have designed strips of rafts as continuous beams limiting the spread of loads to a small zone on either side of the column lines. The contact pressure distribution under the raft and the loads that are causing the pressure are both dependent on the relative settlements. The relative settlements may be ignored only where the foundation system can be assumed to be rigid or the supporting ground is very stiff. In such cases the loads transmitted from the superstructure may be deemed to be unmodified by settlements. Complete soil-structure interaction may be accounted for by treating the subsoil as part of the structure.

Beams on an elastic foundation

One simplification is to model the raft as series of interconnected beams on an elastic foundation. Using Figure 6.1 as reference, the classical governing equation for plates loaded normal to its plane may be modified to include the upward pressure from the soil and may be stated as follows:

$$EI\,(d^4w/dx^4) = qb = k_s bw,$$

where

- E = elastic modulus of the material of the beam (i.e. concrete)
- I = moment of inertia of the beam
- q = load intensity of the beam acting downwards
  Note that by definition the ground pressure $q = w\,k_s$ i.e. the settlement multiplied by modulus of sub-grade reaction.
- k_s = modulus of sub-grade reaction (MN/m^2/m)
  (This 'spring constant' is a parameter used in rigid pavement design and estimates the support provided to the slab by the underlying layers as illustrated in Figure 6.2. Simply, it is pressure/deformation. Some typical values are given in Table 6.1).
- w = deflection of the beam
- b = breadth of the beam

The equation may be solved using a finite difference technique which is not suited to hand calculations. Although useful influence coefficients are presented in some books (e.g. Ray[44]), the calculation process will be tedious in all but very simple cases.

Figure 6.1
Idealised soil structure model.

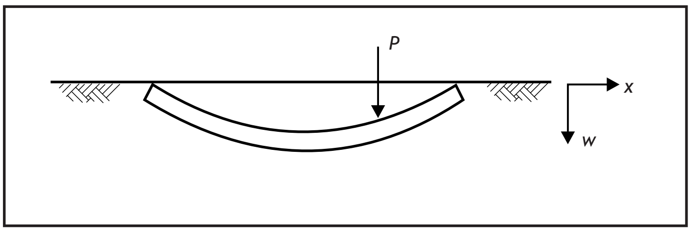


Figure 6.2
Relationship between load, deflection and modulus of subgrade reaction.

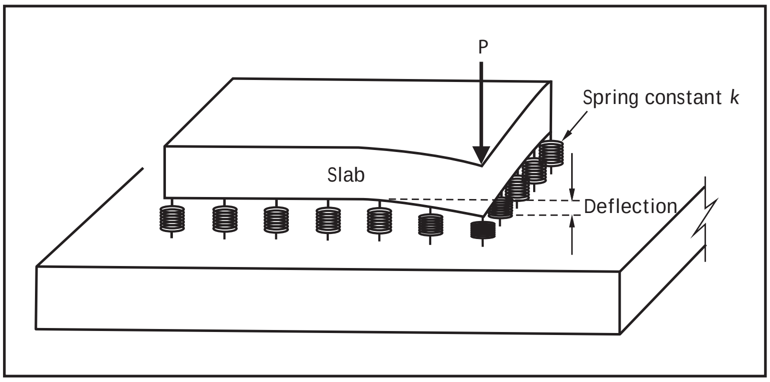


Table 6.1
Typical values of modulus of subgrade reaction, k_s

Soil type	Condition	k_s range MN/m²/m	Typical design value k_s MN/m²/m
I Rock			N/A
II Gravel, sand	Dense	100 to 150 Dense	40
III Clay, Sandy clay	Stiff	20 to 40 Stiff	30
IV Clay, Sandy clay	Firm	10 to 20 Firm	15
V Sand, Silty sand, Clayey sand	Loose	10 to 25 Loose	20
VI Silt, Clay, Sandy clay, Silty clay	Soft		10
VII Silt, Clay, Sandy clay, Silty clay	Very soft		5

Another method that is often used in practice is to represent the soil as series of discrete springs to model the soil-structure interaction. However, the determination of the stiffness of the springs will involve collaboration between the structural engineer and geotechnical specialist. An iterative procedure is commonly adopted. The geotechnical specialist will take into account that the soil extends beyond the structure and will provide realistic values for spring stiffness. Once the correct spring stiffnesses have been agreed they can be directly used in conjunction with a finite element package for the design.

Finite element analysis

It is modern practice to use a finite element package for the design of rafts, where the modulus of elasticity of the soil, E_s, the shear modulus, G, and Poisson's ratio are the principal elastic properties of interest. This is illustrated by the simple raft shown in Figure 6.3. The effect of varying the stiffness of the raft and that of the soil using elastic moduli, E_s, of 75 000, 150 000 and 225 000 kN/m² ('75, 150 and 225 MPa) has been computed using a finite element analysis program. The results are shown in Figures 6.4 to 6.6.

For this example, dome-shaped deflection profiles are realistic and generally will be obtained in full soil-structure models.

Figure 6.3
A simple raft.

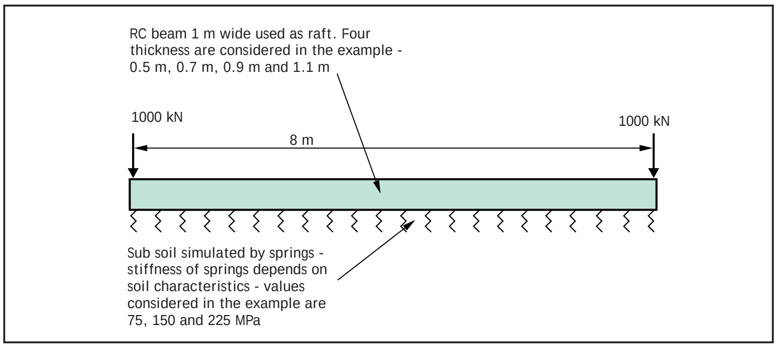


Figure 6.4
Settlement for a simple raft assuming an elastic modulus of E_s = 75 MPa.
Typical for clay with c_u = 100 kN/m²: stiff clay/ medium dense sands and gravels).

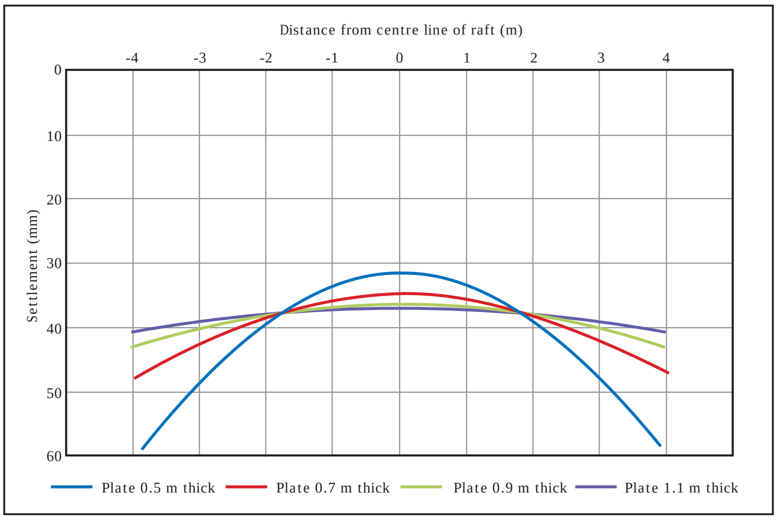


Figure 6.5
Settlement for a simple raft assuming modulus of E_s = 150 MPa
(Typical for clay c_u = 200 kN/m²: very stiff clay/ dense sands and gravels).

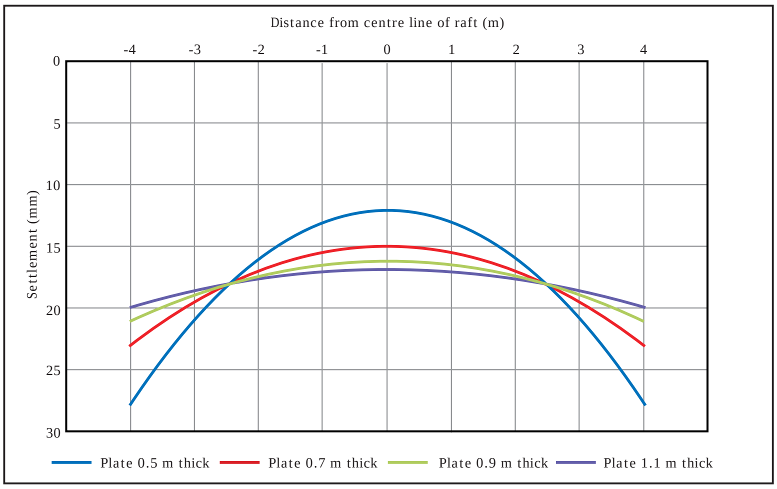
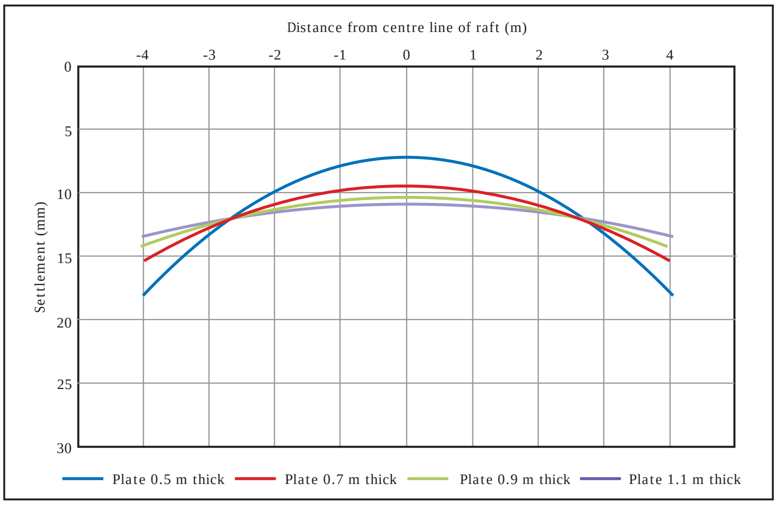


Figure 6.6
Settlement for a simple raft assuming modulus of E_s = 225 MPa
(Typical for clays c_u = 300 kN/m²: hard clay/ very dense sands and gravels).



General conclusions

The following general conclusions can be drawn:

- For a given soil, increasing the stiffness of the raft reduces the peak settlements and the pressures are distributed more evenly.
- In this example, even with the stiffest beam employed, the distribution of settlements (and hence pressure) are far from uniform.

6.2 Structural options for external basement walls

In the permanent condition, basement walls are invariably propped by floor construction. Even where large openings occur adjacent to the wall (e.g. ventilation shafts and light wells) it will be possible to prop the wall at intervals against the floor. The floor diaphragm will need to distribute the reactions from opposite walls by strut action. Generally this is not a problem in concrete floors unless there are large openings in the floor. Generally lateral pressures on opposite faces of the basement will be self-equilibrating. Where there is asymmetry (e.g. where depth of soil retention is unequal), out of balance forces may be transmitted through shear walls and base slab to the ground by friction/adhesion between the walls and the soil behind.

The effects of construction method and sequence must be considered to check if a more critical condition could occur during construction when the wall may not be propped.

Where temporary works such as contiguous or secant piled walls, are required to be incorporated into the permanent works, the type and size of the piles will usually be dictated by preferred methods of work. Issues such as whether the wall will be propped or unpropped, tolerances, availability of plant and whether ground conditions are such that groundwater needs to be cut off all have their influence. The contractor will therefore have input into the final design.

Preliminary designs are often made likening the temporary works to a wall or by using proprietary software. Design of piled wall tends to be an iterative process but final design should be undertaken by specialists once all the design and construction parameters are agreed.

These will include:

- The requirement for the basement and thus grade of basement required
- The depth of the basement
- The nature of the soils and the environs to the project
- The construction sequence and method of excavation
- Any temporary and/or permanent propping requirements.

6.3 Loads to be considered in design

Loads that can occur during construction as well as those on the completed structure should be carefully assessed. Typically the following loads should be considered.

Basement slabs:
- Downward loads from columns and walls.
- Loads on basement slab.
- Upward water pressure (and the effects of floods as appropriate).
- Heave effects.

External walls:
- Lateral earth pressure.
- Lateral water pressure (and the effects of floods as appropriate).
- Lateral pressures that are caused by placing and compacting backfill.
- Lateral pressures caused by surcharge from loads at surface level such as traffic and loads from foundation of adjoining structures.
- Vertical loads and moments imposed by superstructure.
- Lateral loads from superstructure above or from imbalance of lateral pressures.

Generalist engineers should normally be able to establish most of the above loads in conjunction with a competent site investigation report. However, matters such as estimation of heave may well require specialist input. As an initial estimate, instantaneous elastic rebound of the soil, often assumed to be 50% of the total heave effect, is usually removed during excavation. Drained heave, swelling due to intake of water into clay, may take years to manifest itself but may call for measures such as tension piles.

Prescriptive methods allowed under Eurocode 7 [11] are generally not relevant to basement designs.

6.4 Design ground water pressure

The determination of water pressure on basement slabs and walls is important.

Eurocode 7[11] alludes to ground water in several clauses:
- Cl. 2.4.6.1(6) states that the ground water pressures:
  - for ULS represent the most unfavourable values that could occur during the lifetime of the structure;
  - for SLS the design values shall be the most unfavourable values, which could occur during normal circumstances.
- Cl. 2.4.6.1(8) states that the ground water pressure may be derived by applying partial factor to characteristic water pressure or by applying a safety margin to the characteristic water level.
- Cl. 2.4.6.1(10) states that unless the adequacy of the drainage system can be demonstrated and its maintenance ensured the design ground water table should be taken as the maximum possible level, which may be the ground surface.
- Cl. 9.6(3) states that when retaining earth of medium or low permeability (silts or clay) the water table should be assumed to be above the formation level. Unless a reliable drainage system is installed or infiltration is prevented, the values of water pressure shall correspond to a water table at the surface of the retained material.

If the equilibrium level of the water table is well defined and measures are taken to prevent it changing during heavy rain or flood, design water pressures can be calculated from the position of the equilibrium water table, making due allowance for possible seasonal variations. The designer should apply a safety margin to the most unfavourable water level, which could occur during normal circumstances. The assessment of margin should take into account natural variations in the water table, effects of the proposed structure on the water table level, provision of effective drainage, and drainage characteristics of any fill and drainage layers provided. Otherwise, the most adverse water pressure conditions that can be anticipated should be used in design.

In clay soils the equilibrium water table can be established only from piezometric readings over an adequate length of time. CIRIA publication C580[39] provides guidance on the method of establishing water pressures. Flow through clay soils behind a wall becomes irrelevant if water, from above or from trenches, can get into the porous backfill.

The net effect of all the above clauses taken together is that in the majority of projects the permanent works designer will be forced to consider the water table at ground surface level. As shown in Figure 8.1, it is therefore recommended that water levels are considered at both:

- a 'normal' level and
- a maximum level i.e. either 'normal ' level plus a safety margin or, more usually, to the surface of the retained material. An even higher level may need to be considered if flooding is expected and is to be prevented in the basement.

Commonly, it may not be possible to establish long-term ground water levels, in which case maximum ground water levels should be used. Appropriate design ground water levels and partial load factors for these conditions are discussed in Section 8.2 and Chapter 9.

6.5 Unplanned excavations

In the design of cantilever or propped cantilever retention systems (e.g. contiguous piles or diaphragm walls), allowance should be made for possible unplanned excavations or softening of soils in front of the walls.

This should be:

- not less than 0.5 m; and
- not less than 10% of the total height retained for cantilever walls or of the height retained below the lower support block for propped or anchored walls.

7. Calculation of lateral earth pressures

There are various methods of calculating lateral earth pressures. This section looks at relatively simple equilibrium hand calculation analysis for statically determinate cases. More sophisticated soil-structure interaction and/or finite element analyses may be more appropriate for large or complex structures.

The key data required are the density and strength parameters of the soil and these are discussed below. Recommended design ground water levels are shown in Figure 8.1. The effect of ground water is discussed in Section 7.3.2.

7.1 Density of soils

Typical values for mass density for granular and cohesive soils are given in Table 7.1. These may be used for preliminary design in the absence of test data for the site[15].

Table 7.1
Mass densities of soils.

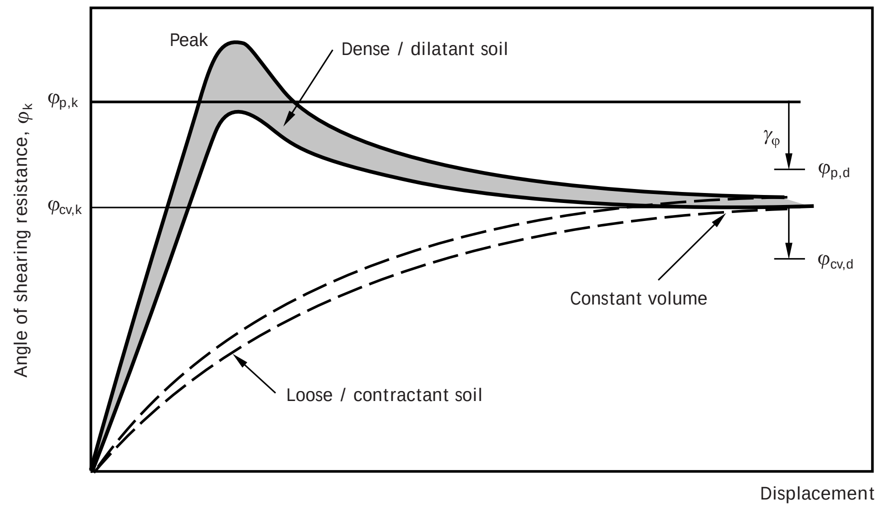
Soil type	Moist bulk density, γ (kg/m^3)		Saturated bulk density, γ (kg/m^3)	
	Loose	Dense	Loose	Dense
Granular				
Gravel	1600	1800	2000	2100
Well graded sand and gravel	1900	2100	2150	2300
Coarse or medium sand	1650	1850	2000	2150
Well graded sand	1800	2100	2050	2250
Brick hardcore	1300	1750	1650	1900
Cohesive				
Soft clay	1700		1700	
Firm clay	1800		1800	
Stiff clay	1900		1900	
Stiff or hard glacial clay	2100		2100	

7.2 Strength parameters for soils

Selection of design values for strength parameters requires care. Conventional methods of establishing active and passive pressures using Rankine or Coulomb methods implicitly assume plasticity in the soil (i.e. the soil is at point of failure). The laboratory tests establish the maximum values of parameters. However, under serviceability conditions deformations are likely to be comparatively small. The soil will therefore operate at below peak strength conditions. This has a crucial bearing on the design values to be used. Under the ultimate limit state of the soil, when the deformations will be relatively large, the soil will operate beyond peak strength conditions. The soil may dilate and deformations may increase so that strengths approach critical state values consistent with the strength envelope for loose or normally consolidated soils. See Figure 7.1.

BS 8002[15] provides some empirical relationships that can also be used in designs in conjunction with Eurocode 7[11]. These are reproduced below in Sections 7.2.1 and 7.2.2.

Figure 7.1
Peak and constant volume angles of shearing resistance[45]



Notes

$\varphi_{p,k}$, ($\varphi_{cv,k}$): Characteristic peak (constant volume) angle of shearing resistance
φ_{pd}, ($\varphi_{cv,d}$): Design peak (constant volume) angle of shearing resistance

7.2.1 Granular soils

In granular soils the strength property that is of interest is the angle of shearing resistance, φ'.

Estimated peak effective angle of shearing resistance,

$\varphi'_{max} = 30 + A + B + C$

Estimated critical state angle of shearing resistance,

$\varphi'_{crit} = 30 + A + B$ which is the upper limiting value.

Parameters A, B and C allow respectively for angularity of particles, grading of sand and gravel, and results of standard penetration tests (SPT). The blow count N' is the modified SPT value (see BS 8002 for the modification curve).

Table 7.2
Values of A, B and C for granular materials[15]

Angularity	A (degrees)	Grading	B (degrees)	Number of blows, N'	C (degrees)
Rounded	0	Uniform	0	**<10**	0
Sub-angular	2	Moderate grading	2	**20**	2
				40	6
Angular	4	Well graded	4	**60**	9

Note
The practice of 'correcting' the SPT blow count N by a factor of up to 2, or more, is debatable – it largely originated from 'correcting' early empirical charts for the bearing capacity of coarse soils. Take geotechnical advice before using a corrected N' value in Table 7.2.

7.2.2 Clay soils

There are complications with clay soils. Many clays in the UK are overconsolidated and their strength is represented by one of two idealisations, namely:

- predominantly frictional or
- purely cohesive behaviour.

Frictional behaviour corresponds to conditions where pore water pressures can be defined or assumed and thus allow the strength of the soil to be characterised in terms of effective stress. Cohesive behaviour refers only to the immediate short term when pore water pressures cannot be conveniently defined or assumed. Then, the strength of the soil is represented by the original undrained shear strength, c_u.

Table 7.3
Comparison of cohesive and frictional models for clay.
Taken from CIRIA Report 114[46]

	Cohesive (total stress theory)	Frictional (effective stress theory)
Characterisation	Undrained shear strength c_u	Strength proportional to tan φ' and normal effective stress (a cohesion intercept may also be included)
Application	Relevant only to immediate short-term conditions. Restricted to temporary works	Any condition (temporary and permanent). Essential for permanent works design
Limitation	Total stress theory relies on undrained behaviour and so has to be used with caution as softening can proceed much faster than might be expected	Effective stress theory requires a knowledge of pore water pressures within the soil
Conservatism	Intrinsically unsafe for overconsolidated clays	For temporary works the method is conservative (if the fact that the pore water pressures are less than the final equilibrium values is ignored)
Difficulties	The value of c_u is test-dependent and is influenced by sample disturbance and sample size	Any cohesion intercept (c') is difficult to assess from laboratory tests

In saturated soil, pore water pressure increases initially on application of load. A hydraulic gradient is set up and water flows out and pore water pressures gradually move toward their long-term equilibrium value. This is referred to as consolidation and it is time dependent. Time taken for consolidation increases with decreasing stiffness and decreasing permeability. In sands and gravel it is almost instantaneous.

In clays consolidation (or swelling) takes a very long time. Loading will cause long term consolidation i.e. failure is more likely in the short term immediately after loading. However, unloading (e.g. during excavation) will promote swelling and softening of the soil, i.e. failure is more likely in the medium or long term. Therefore in clays both possibilities will need verification. Long-term calculations are carried out using effective stress parameters – drained analysis. Short-term calculations use total stress parameters – undrained analysis.

Traditionally, many generalist engineers have used the cohesion model and total stress procedures. However, any tendency for the pore water to drain will lead to changes of voids ratio and hence the value of undrained cohesion. Voids in soil are not fixed in place; water can move and voids can collapse or expand. Therefore undrained cohesion

values are difficult to predict with certainty. In the long term, clays behave as granular soils exhibiting friction and dilation.

For this reason calculations to predict long-term values of pressures are undertaken using the effective stress method as illustrated in Sections 7.3 and 7.4. A comparison of the two idealisations is shown in Table 7.3.

When using the effective stress method the friction and cohesion can be characterised by the two parameters angle of shearing resistance, φ' and cohesion intercept in terms of effective stress, c' (rather than peak angle of shearing resistance, φ'_{max}). In the absence of test data BS 8002 [15] recommends values of φ' that can be used with $c' = 0$ and these are shown in Table 7.4.

Table 7.4
Values of φ' for clay soils.

Plasticity index (%)	15	30	50	80
φ' (degrees)	30	25	20	15

Geotechnical advice should be obtained before using 'cohesion' in terms of effective stress c' as well as φ' in an analysis.

7.2.3 Silts

It is difficult to provide general data for silt. The designer should consult a specialist for guidance.

7.3 Earth pressures – characteristic active, passive and at rest pressures

For all the reasons given above, for the design of basement structures it will be generally satisfactory to treat the soil as granular and apply the appropriate earth pressure formulae. Expressions for *active* and *passive* pressures are given below in terms of effective stress. Earth pressure caused by surcharge loading is discussed in Section 7.4.

7.3.1 Design value of effective angle of shearing resistance, φ'_d

In Eurocode 7 calculations are carried out using design values of the effective angle of shearing resistance φ'_d, which are obtained as follows:

$$\tan \varphi'_d = \tan (\varphi'_k/\gamma_\varphi)$$

where

φ'_k $= \varphi'_{max}$ for granular soils and
$= \varphi'$ for clay soils, Sections 7.2.1 and 7.2.2
φ'_{max} and φ' are as defined in Section 7.2.1 and 7.2.2

γ_φ = 1.0 or 1.25 dependent on the *combination* being considered. See Section 8.2

As is apparent, the value of φ'_d depends on the partial factor for the soil parameter γ_φ. So for 'Combination 1' and 'Combination 2' (see Section 8.2), two values of φ'_d must be considered. Values for φ'_d may be found for different values of φ' in Table A2 of this publication.

In granular soils φ'_d should be limited to not more than φ'_{crit} as defined in Section 7.2.

7.3.2 Active and passive pressures

Generally for bottom up construction, active earth pressures using coefficient K_{ad} will be appropriate.

Active pressure

Active pressure at depth *z* below ground surface

$$\sigma'_{ah} = K_{ad}\,\sigma'_{v} + u$$

Passive pressure

Passive pressure at depth *z* below ground surface

$$\sigma'_{ph} = K_{pd}\,\sigma'_{v} + u$$

where

K_{ad} = active earth pressure coefficient (see Table A3)
K_{pd} = passive earth pressure coefficient (see Table A3)

Values for K_{ad} and K_{pd} may be found for different values of φ'_{d} in Annex C of Eurocode 7[11] or in Table A3 of this publication.

σ'_{v} = vertical effective stress
$= \int \gamma\,\delta z + q - u$

where

$\int \gamma \delta z$ = integral of $\gamma \delta z$ carried out from the surface to depth *z*
γ = weight density of the soil (or soils in multi-layer soils)
δz = incremental depth below the surface
q = uniform surcharge at the ground surface

u = pore water pressure at depth *z*
$= \gamma_{w}\,(z - z_{w})$ for hydrostatic (no seepage) conditions

where

γ_{w} = density of water
z_{w} = depth of water table below the surface

7.3.3 At rest pressure coefficients

In top down construction, there is unlikely to be any significant movement. In such, and similar conditions the appropriate value of earth pressure is the pressure *at rest* derived by substituting K_{0d} in place of K_{ad} such that at rest pressure at depth *z* below ground surface

$$\sigma'_{0h} = K_{0d}\,\sigma'_{v} + u$$

where

$K_{0d} = (1 - \sin \varphi'_{d})$ for loose and lightly consolidated soils and

$K_{0d} = (1 - \sin \varphi'_{d})\,(\mathrm{OCR})^{0.5}$ for over-consolidated soils

where

OCR = over-consolidation ratio for clay soils, which should be determined by tests. In the absence of other data OCR may be taken as 3 as a reasonable value for preliminary design

φ'_d is defined in Section 7.3.1 above or see Table A3.

The above values for K_{0d} apply when the ground surface is flat. On sloping ground:

$$K_{0d,\beta} = K_{0d,flat}\,(1 + \sin\beta)$$

where

β is the inclination of the surface to the horizontal.

Values of earth pressures obtained using the procedures noted above are the most unfavourable that are likely to occur. Whilst they occur at working conditions, they should be used in calculations for both ultimate and serviceability limit states of the structure. For ULS calculations for the permanent case, it is generally convenient to calculate the lateral pressures using unfactored values of actions and multiply the results by the appropriate partial factor for permanent or variable actions.

It should be noted that considerable movement can occur below excavation level and full analysis should be undertaken. Also granular soils can also be overconsolidated.

7.4 Earth pressure caused by surcharge loading

Surcharge can arise from a number of sources including:

- loads from adjacent roads railways buildings and so forth;
- loads due to construction activities;
- variations in surface levels in undulating ground.

Imposed loads: general

In the design of walls in basements with depth greater than 3 m, a minimum surcharge of 10 kN/m^2 should be assumed. For shallower basements the surcharge may be reduced if the designer is confident that a surcharge of 10 kN/m^2 will not occur during the life of the structure. These values based on BS 8002[15] may be used for both ultimate and serviceability limit states.

Imposed loads: highways

Where basement walls give support to highways it was traditional to check for the effects of HA loading[47] at ground level. The equivalent to BS EN 1991-2 and the UK NA[48, 48a] is the application of the load model in Figure NA.6 of the National Annex[48a], where the axle loads are 65, 65, 115 and 75 kN (Σ = 320 kN) at centres 1.2, 3.9 and 1.3 m apart (Σ = 6.4 m). Each axle consists of two wheels of equal load at a distance of 2.0 m apart.

Clause 6.6.3 of PD 6694-1[49] allows an alternative. For global effects on 'other earth retaining walls' adjacent to highways, two vertical uniformly distributed transverse line loads of Q_L, are applied 2.0 m apart on a notional lane of the carriageway,

where

$Q_L = 320/(2 \times 6.4) = 25$ kN/m over a length of 6.4 m.

Beside normal γ_Q factors, axle loads and line loads are subject to an overload factor of 1.5 and, when checking a single vehicle in one notional lane, a dynamic factor of 1.4. The dynamic factor dissipates to 1.0 at 7.0 m depth or when convoys of vehicles are considered in each notional lane (representing a traffic jam situation).

In association the axle loads or line loads, it is suggested that a surcharge of 5 kN/m^2 is applied as an imposed load to pavements adjacent to basements. This figure is in agreement with NA.2.36 of the UK NA to BS EN 1991-2[48] as a maximum uniformly distributed load for continuous dense crowding (e.g. footbridges serving a stadium) and with BS EN 1991-1-1 for traffic and parking areas with vehicles > 30 kN.

Lateral pressures caused by surcharge loading

Lateral pressures caused by surcharge loading may be calculated using the procedures shown below for the different types of surcharge. Elastic vertical stress in the soil at the location is multiplied by the appropriate earth pressure coefficient to obtain the lateral pressure.

Other surcharge loadings

Loads from adjacent buildings should be represented by surcharge loads at the depth of their foundations. Surcharge loads from roads and railways may be determined from the relevant part of Eurocode 1.

Wherever possible stockpiles should not be placed adjacent to retaining walls.

Walls that are constructed before the placement of backfill should be designed to withstand the effects of forces caused by compaction. Loads caused by compaction equipment should be established for each project. Some typical loads are indicated in Table 7.5 and Section 7.5.5 shows how these loads may be converted into characteristic design pressures.

7.4.1 Uniformly distributed surcharge

Lateral thrust due to uniformly distributed surcharge (q kN/m^2) may be calculated by considering the surcharge pressure as an initial overburden pressure at ground level. Lateral pressure may be taken as:

$$\sigma'_{ah} = K_h \, q$$

where

K_h $= K_{ad}$ or K_{0d} as appropriate (see Table A3)

q = surcharge load, kN/m^2

Generally, it will be satisfactory to use the same value of K_h as that used for the permanent action. This is however, an area of contention. As surcharge loading is applied to theoretically undisturbed soil some argue that K_0 should be used: recognising that construction operations have an effect, many use $(K_{ad} + K_{0d})/2$ for surcharge loads and, in propped construction, for permanent actions. Engineering judgement is required to assess the effects of construction and rigidity of the wall to determine an appropriate

value of K_h. BS 8002[15] suggests that in cohesionless soils movement in the order of only 10^{-3} radians is required to reduced earth pressures to their fully active values. In this publication the value of K_h used is that for the permanent action.

7.4.2 Point load (to be read in conjunction with Figure 7.2)

Using Boussinesq[49] equations to determine vertical pressure, lateral pressure at a point O on a wall due to a discrete load Q may be taken as:

$$\sigma'_{ah} = K_h (3QZ^3)/(2\pi R^5)$$

where

$K_h = K_{ad}$ or K_{0d} as appropriate (see section 7.4.1 and Table A3)

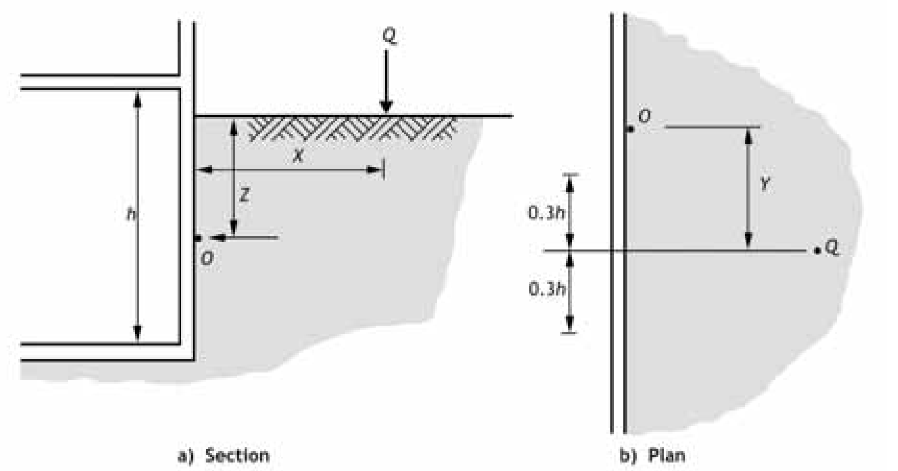
Q = load, kN

Z = depth, m

$R = (X^2 + Y^2 + Z^2)^{0.5}$

It is recommended that horizontal earth pressures against 'rigid' walls determined using Boussinesq's theory of stresses in an elastic half space should be doubled for design purposes. Boussinesq's theory for horizontal pressures assumes horizontal movement. However, with truly rigid walls there is actually no movement. So an identical balancing surcharge on the other side of the wall, i.e. a mirror image surcharge, is required and this in effect doubles the pressure[78]. Doubling pressure is in line with field data by Terzarghi and with French practice.

Figure 7.2
Definitions for calculating lateral pressures due to surcharge loading (point load or line load parallel to wall).



O is the point at which the lateral pressure is sought

Point load
Load Q in kN
Parameter $R = (X^2 + Y^2 + Z^2)^{0.5}$
Normally design 0.6h length of wall centred on the point load for the total load on this section

Line load parallel to wall
Load Q kN/m
Parameter $R = (X^2 + Z^2)^{0.5}$

The pressure will be at the maximum when $Y = 0$ (see Figure 7.2b) and it will reduce at other locations depending on its distance from the load. It will normally be sufficient to design the length of wall equal to 0.6*h* centred on the point load (as shown in Figure 7.2b) for the total load on this section.

7.4.3 Line load parallel to wall (to be read in conjunction with Figure 7.2)

Assuming that the length of the load is comparable to that of the wall, lateral pressure at a point O will depend mainly on the depth *z*. In this case lateral pressure may be taken as:

$\sigma'_{ah} = K_h\,(2QZ^3)/(\pi R^4)$.

where

K_h = K_{ad} or K_{0d} as appropriate (see Section 7.4.1 and Table A3)

Q = load per metre length of load kN/m

Z = depth, m

$R = (X^2 + Z^2)^{0.5}$

As explained in 7.4.2, it is recommended that horizontal earth pressures determined using Boussinesq's theory of stresses in an elastic half space should be doubled.

7.4.4 Strip load parallel to wall (to be read in conjunction with Figure 7.3)

Assuming that the length of the load is comparable to that of the wall, lateral pressure at a point O will depend only on the depth *z*. In this case lateral pressure may be taken as:

$\sigma'_{ah} = K_h\,(q/\pi)\,[\alpha + \sin\alpha\cos(\alpha + 2\beta)]$

where

K_h = K_{ad} or K_{0d} as appropriate (see Table A3)

q = load per metre width of load

α, β = angles defined in Figure 7.3 (radians)

As explained in 7.4.2, it is recommended that horizontal earth pressures determined using Boussinesq's theory of stresses in an elastic half space should be doubled.

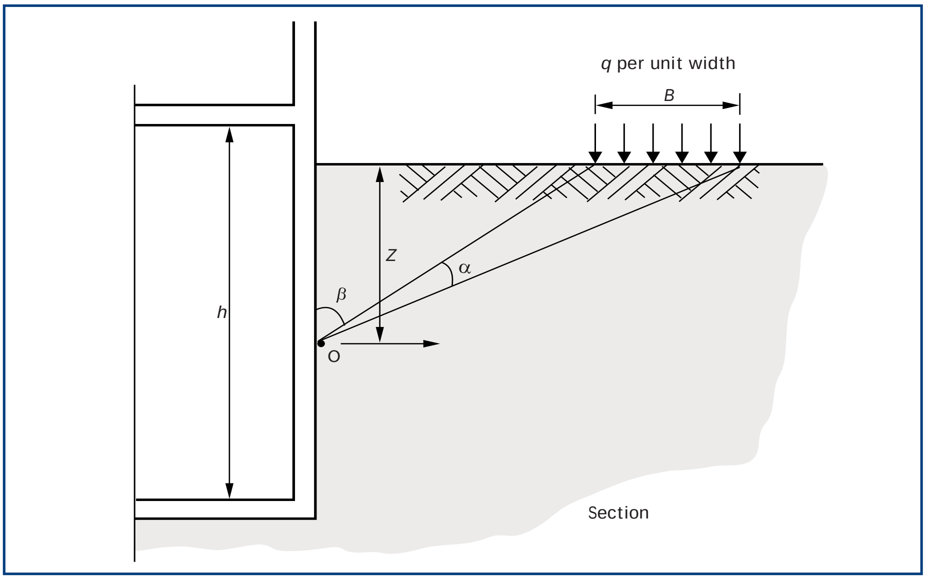


Figure 7.3
Definitions for calculating lateral pressures due to surcharge loading (strip load parallel to wall).

7.4.5 Rectangular loads exerting uniform pressure

Standard text books provide solutions for vertical stress, $\sigma_{v,z}$, at depth *z under a corner of a rectangular area* carrying a uniform pressure *q*. It is usually in the form

$\sigma'_{v,z} = qI_r$

where

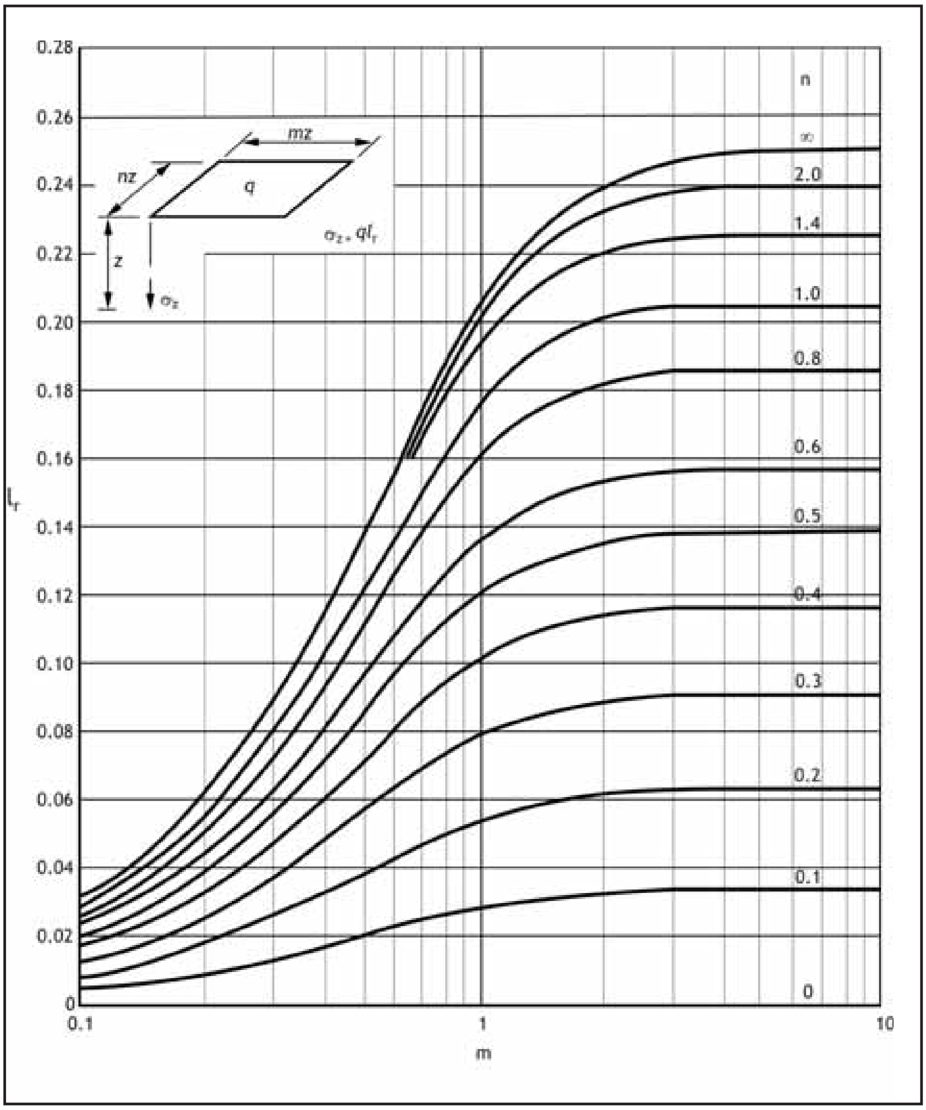
q = uniform pressure

I_r = coefficient. Values of I_r are provided for different aspect ratios of the loaded area to depth. One such chart is reproduced as Figure 7.4 from Craig[50].

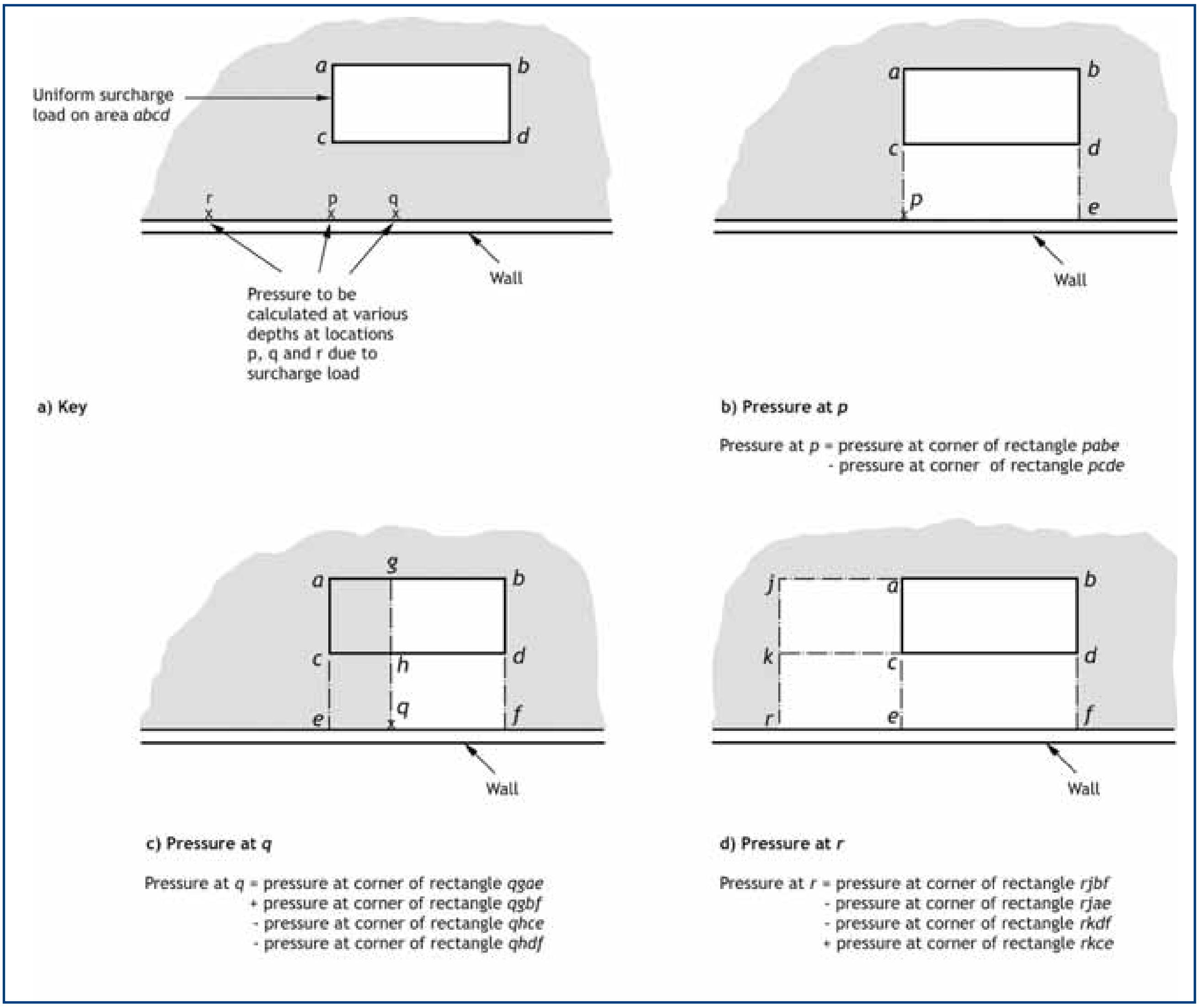
Lateral pressure at the required depth may be determined as $K_h qI_r$

As explained in 7.4.2, it is recommended that horizontal earth pressures determined using Boussinesq's theory of stresses in an elastic half space should be doubled.

Figure 7.4
Fadum's solution for vertical pressure under a corner of a rectangular area carrying uniform pressure[50].



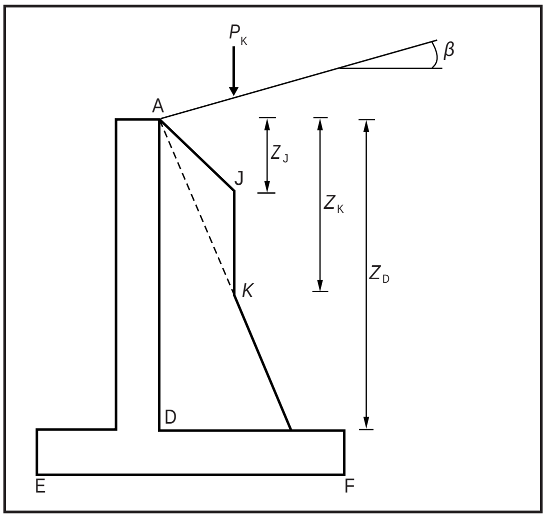
The method of superposition allows the determination of vertical stress under any point *within or outside* the loaded area. This is illustrated in Figures 7.5b to 7.5d for determining pressures caused by a rectangular loaded area shown in Figure 7.5a. As illustrated, the effects of patch loads are derived by calculating pressures as if the load extends to the wall, and then subtracting the pressure for the patch load that does not exist.



Above: **Figure 7.5**
Definitions for calculating lateral pressures due to uniform surcharge load on a rectangle.

7.4.6 Compaction pressures

Walls that are constructed before the placement of fill behind them should be designed to withstand the effects of compaction pressures. A practical approach originally developed by Ingold[51] is commonly used. The pressure distribution to be used in design is indicated in Figure 7.6.



Right: **Figure 7.6**
Definitions for calculating compaction pressures on walls.

Line AJ represents passive pressures arising from compaction pressures and line AK represents active pressures arising from the contained ground. Lateral pressure at depth z_i is calculated using the following definitions:

Between A and J the horizontal pressure is $K_{pd}\ \gamma_{k,f}\ z$
Between J and K the horizontal pressure is $K_{pd}\ \gamma_{k,f}\ z_J$
Between K and D the horizontal pressure is $K_{ad}\ \gamma_{k,f}\ z$
where

K_{pd} = passive pressure coefficient (see Table A3)
K_{ad} = active pressure coefficient (see Table A3)
$\gamma_{k,f}$ = characteristic value of the density of the fill material and
z = depth from surface $\leq Z_J$ or $\geq Z_k$
where

Z_J = depth from surface to point J
$= (1/K_{pd})\ (2\ P_d/\pi\ \gamma_{k,f})^{0.5}$
Z_K = depth from surface to point K
$= (1/K_{ad})\ (2\ P_d/\pi\ \gamma_{k,f})^{0.5}$
$= (K_{pd}\ /K_{ad})\ z_J$
where

$P_d = \gamma_F\ P_k$
where

γ_F = partial factor (= γ_G assumed). See Table 8.2 and
P_k = characteristic design force in Table 7.5

Table 7.5
Design force due to typical compaction equipment[52, 53].

Compaction equipment	Mass (kg)	Centrifugal vibrator force (kN)	Characteristic design force (kN)
80 kg vibrating plate compactor	80	14	14.3
180 kg vibrating plate compactor	180	80	81.8
350 kg pedestrian operated vibrating roller	350	12	15.4
670 kg vibrating plate compactor	670	96	102.6
1.5 t pedestrian operated vibrating roller	1530	58	73.0
2.8 t smooth wheeled roller	2800	N/A	27.5
6.9 t double vibrating roller	6900	118	185.7
8.6 t smooth wheeled roller	8600	N/A	84.4

7.5 Examples of calculating lateral pressures

Examples of the following calculations are given:

- characteristic active pressures
- at rest pressures
- surcharge loading from
  - imposed loads
  - a pad foundation and
  - compaction.

As noted earlier, lateral earth pressures on retaining structures are obtained using design values of the effective angle of shearing resistance φ'_d. However, the value of φ'_d depends on the partial factor for the soil parameter γ_φ. BS EN 1990 and the National Annex to BS EN 1997-1[11] require the consideration of two separate combinations of loads (i.e. two sets of partial factors). These partial load factors are associated with partial factors on soil properties leading to ‘Combination 1’ and ‘Combination 2’ factors. (See Section 8.2). So for the two combinations to be considered γ_φ varies. Thus the partial factor for material governs the design value of the load!

In design calculations values of φ'_d should be established for each combination.

7.5.1 Example: active pressures

Calculate the horizontal pressures on the basement wall shown in Figure 7.7. Assume that the basement is constructed bottom up (i.e. pressures will be active pressures).

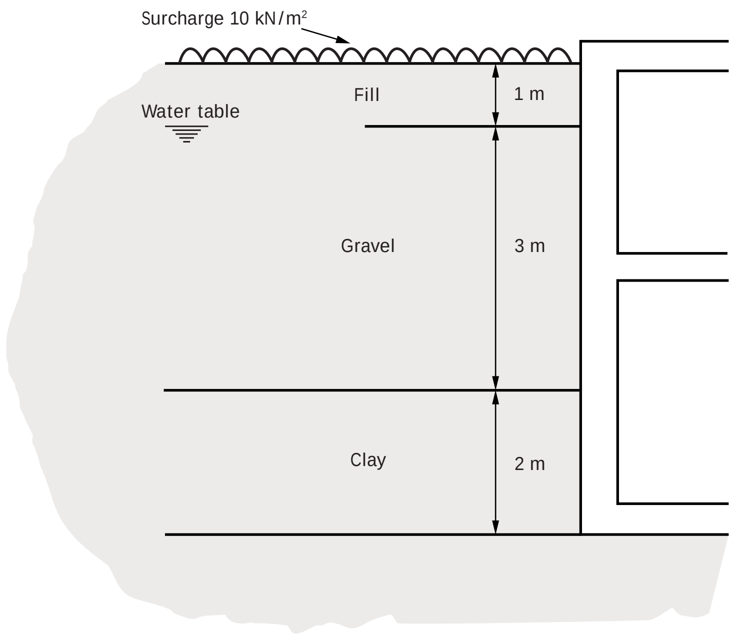


Figure 7.7
Section through basement wall

mpa The Concrete Centre	Project details: Calculations of active lateral earth pressures	Calculated by: CG	Job no.: CCIP - 044
		Checked by: PG	Sheet no.
		Client: TCC	Date

Determine design values of φ'_d

	Combination 1	Combination 2 (and serviceability calculations)
$\gamma_\varphi =$ **Fill material** This is treated as granular material. $\varphi'_{max} = 30 + A + B + C$ (Table 7.2) Conservatively assume that A, B and C = 0 $\therefore \varphi'_{max} = 30°$ and $\varphi'_{crit} = 30°$ $\tan \varphi'_d = \tan \varphi'_k / \gamma_\varphi = \tan \varphi'_{max} / \gamma_\varphi$	1.0	1.25

φ'_d :	$\varphi_{max} = 30°$ $\tan \varphi'_d = \tan \varphi'_k / \gamma_\varphi$ $= \tan \varphi'_{max} / \gamma_\varphi$ $= \tan 30/1.00$ $= 0.577$ $\therefore \varphi'_d = 30° \le \varphi'_{crit}$ $\therefore$ OK use $\varphi'_d = 30°$	$\tan \varphi'_d = \tan \varphi'_k / \gamma_\varphi$ $= \tan \varphi'_{max} / \gamma_\varphi$ $= \tan 30/1.25$ $= 0.577/1.25$ $= 0.462$ $\therefore \varphi'_d = 24.8° < \varphi'_{crit}$ $\therefore$ OK use $\varphi'_d = 24.8°$
Gravel Assume that it is sub-angular, has moderate grading and has N = 20. From Table 7.2, A = 2, B = 2 and C = 2. $\therefore \varphi'_{max} = 30° + 2° + 2° + 2° = 36°$ and $\varphi'_{crit} = 30° + 2° + 2° = 34°$		
$\varphi'_d =$	$\varphi'_{max} = 36°$ $\varphi'_d = 36° \le \varphi'$ $\varphi'_{crit} = 34°$ $\therefore$ use $\varphi'_d = 34°$	$\tan \varphi'_d = \tan 36/1.25$ $= 0.727/1.25$ $= 0.581$ $\therefore \varphi'_d = 30.2° < \varphi_{crit}$ $\therefore \varphi'_d = 30.2°$
Clay Assume that the plasticity index is 15% $\therefore$ From Table 7.4, $\varphi'_d =$	30°	$\tan \varphi'_d = \tan 30/1.25$ $= 0.577/1.25$ $= 0.462$ $\therefore \varphi'_d = 25°$

Determine active pressure

The calculation of pressures for combinations 1 and 2 are shown in Table 7.6 and 7.7 and the results are presented in Figure 7.8. Note that partial factors for actions have not been applied in these tables.

Bearing in mind that only the surcharge load is treated as a variable action, it will be convenient to deduct the values in column 11, the pressure caused by surcharge loading, and then apply the appropriate partial factors to each component.

Table 7.6 Active pressures for Combination 1

	2	3	4	5	6	7	8	9	10	11	12	13	14
Soil layer	Depth below surface, z (m)	Density, γ_e (kg/m^3)	$\int \gamma Z$ (kN/m^2)	Head of water, $z - z_w \geq 0$ (m)	Pore water pressure, u (kN/m^2)	Minimum surcharge, q (kN/m^2)	Effective vertical stress ,σ'_v (kN/m^2) (4) + (7) − (6)	φ'_d (degrees)	$K_h = K_{ad}$	Active horizontal pressure σ'_{ah} (kN/m^2) (8) × (10) + (6)	q_k(kN/m^2) (7) × (10)	q_{kw}(kN/m^2) (= 6)	g_k(kN/m^2) (11) − (12) − (13)
Fill	0	1650	0	0	0	10	10	30	0.33	3.3	3.3	0.0	0.0
	1	1650	16.5	0	0	10	26.5	30	0.33	8.8	3.3	0.0	5.5
Gravel	1	2000	16.5	0	0	10	26.5	34	0.28	7.5	2.8	0.0	4.7
	4	2000	76.5	3	30	10	56.5	34	0.28	46.0	2.8	30.0	13.2
Clay	4	1800	76.5	3	30	10	56.5	30	0.33	48.8	3.3	30.0	15.5
	6	1800	112.5	5	50	10	72.5	30	0.33	74.1	3.3	50.0	20.8

Note
Partial factors for actions have not been applied in the above table.

Similarly:

Table 7.7 Active pressures for Combination 2

	2	3	4	5	6	7	8	9	10	11	12	13	14
Soil layer	Depth below surface, z (m)	Density, γ_e kg/m^3)	$\int \gamma Z$ (kN/m^2)	Head of water, $z - z_w \geq 0$ (m)	Pore water pressure, u (kN/m^2)	Minimum surcharge' q (kN/m^2)	Effective vertical stress σ'_v (kN/m^2) (4) + (7) − (6)	φ'_d (degrees)	$K_h = K_{ad}$	Active horiz. Pressure, σ'_{ah} (kN/m^2) (8) × (10) + (6)	q_k(kN/m^2) (7) × (10)	q_{kw}(kN/m^2) (= 6)	g_k(kN/m^2) (11) − (12) − (13)
Fill	0	1650	—	0	0	10	10.0	24.8	0.41	4.1	4.1	0	0
	1	1650	16.5	0	0	10	26.5	24.8	0.41	10.9	4.1	0	6.8
Gravel	1	2000	16.5	0	0	10	26.5	30.2	0.33	8.7	3.3	0	5.4
	4	2000	76.5	3	30	10	56.5	30.2	0.33	48.6	3.3	30	15.3
Clay	4	1800	76.5	3	30	10	56.5	25.0	0.41	53.2	4.1	30	19.1
	6	1800	112.5	5	50	10	72.5	25.0	0.41	79.7	4.1	50	25.6

Note
Partial factors for actions have not been applied in the above table.

Summary:

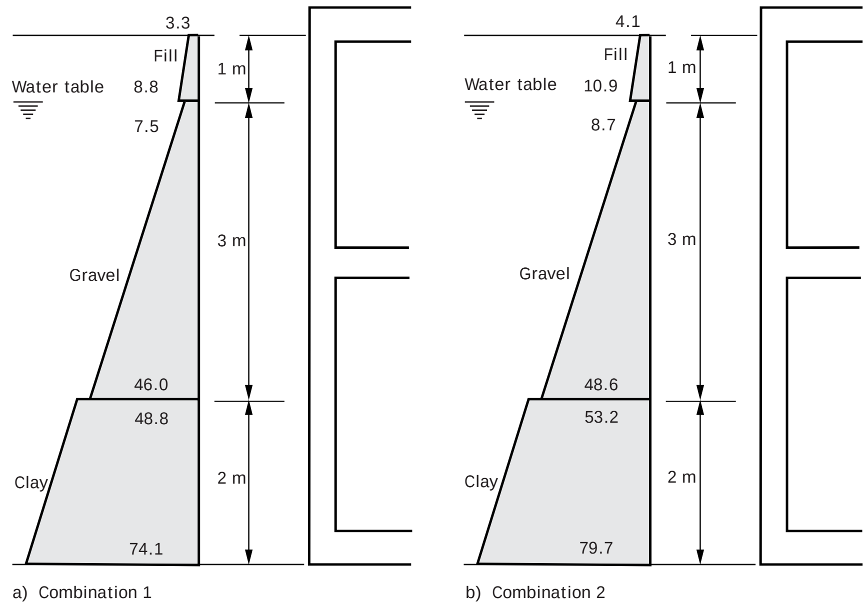


Figure 7.8 Basement wall: characteristic active pressures (kN/m²)

7.5.2 Example: at-rest pressures

Calculate the horizontal pressures on the basement wall shown in Figure 7.7. Assume that the basement is constructed using top down method (i.e. pressure at rest will be relevant).

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Soil properties:

		Combination 1	Combination 2
	γ_φ	= 1.0	= 1.25
Fill material	As before φ'_d	= 30°	= 24.8°
	$K_{0d} = (1 - \sin \varphi'_d)$	= 1 – 0.50 = 0.50	= 1 – 0.419 = 0.581
Gravel	As before φ'_d	= 34°	= 30.2°
	$K_{0d} = (1 - \sin \varphi'_d)$	= 0.441	= 0.497
Clay	As before φ'_d	= 30°	= 25°
	$K_{0d} = (1 - \sin \varphi'_d)\,(OCR)^{0.5}$ (Assuming that OCR = 3.0)	= $(1 - 0.50) \times 3^{0.5}$ = 0.866	= $(1 - 0.419) \times 3^{0.5}$ = 1.00

The calculation of at-rest pressures is shown in Table 7.8 and the results are presented in Figure 7.9. Note the significant increase in pressure compared with active pressures in Example 7.5.1

Table 7.8 At-rest pressures for load combinations 1 and 2

	2	3	4	5	6	7	8	9	10	11	9	10	11
								Combination 1			Combination 2		
Soil layer	Depth below surface Z (m)	Density γ (kg/m^3)	$\int \gamma Z$ (kN/m^2)	Head of water, (m)	Pore water pressure u (kN/m^2)	Minimum surcharge q, (kN/m^2)	Effective vertical stress σ'_v (kN/m^2) (4) + (7) – (6)	φ'_d (degrees)	$K_h = K_{0d}$	Active horizontal pressure, σ'_{ah} (kN/m^2) (8) × (10) + (6)	φ'_d (degrees)	$K_h = K_{0d}$	Active pressure horizontal σ'_{ah} (kN/m^2) (8) × (10) + (6)
Fill	0	1650	—	0	0	10	10.0	30	0.500	5.0	24.8	0.581	5.8
	1	1650	16.5	0	0	10	26.5	30	0.500	13.3	24.8	0.581	15.4
Gravel	1	2000	16.5	0	0	10	25.5	34	0.441	11.7	30.2	0.497	13.2
	4	2000	76.5	3	30	10	56.5	34	0.441	54.9	30.2	0.497	58.1
Clay	4	1800	76.5	3	30	10	56.5	30	0.866	78.9	25.0	1.000	86.5
	6	1800	112.5	5	50	10	72.5	30	0.866	112.8	25.0	1.000	122.5

Note
Partial factors for actions have not been applied in the above table.

Summary:

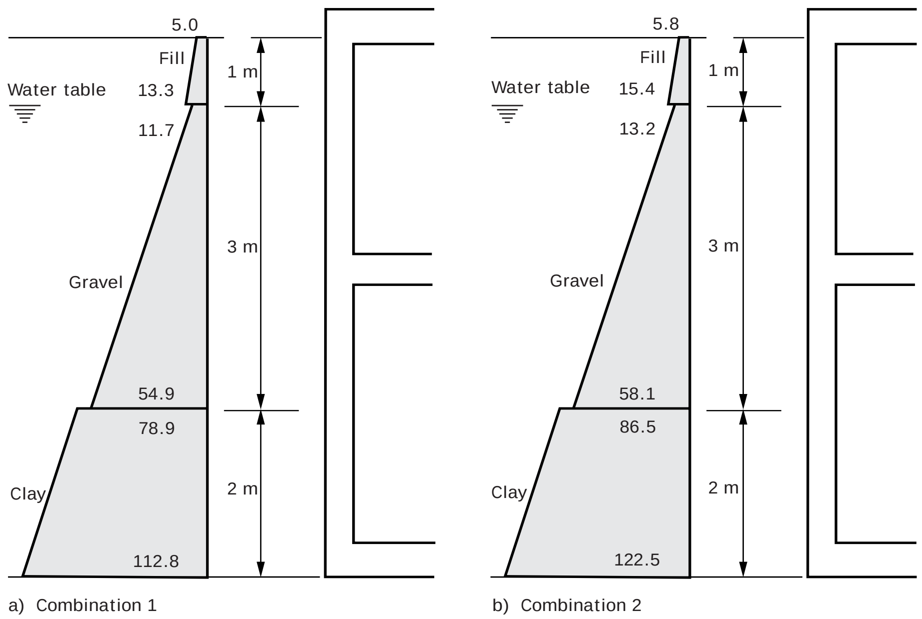


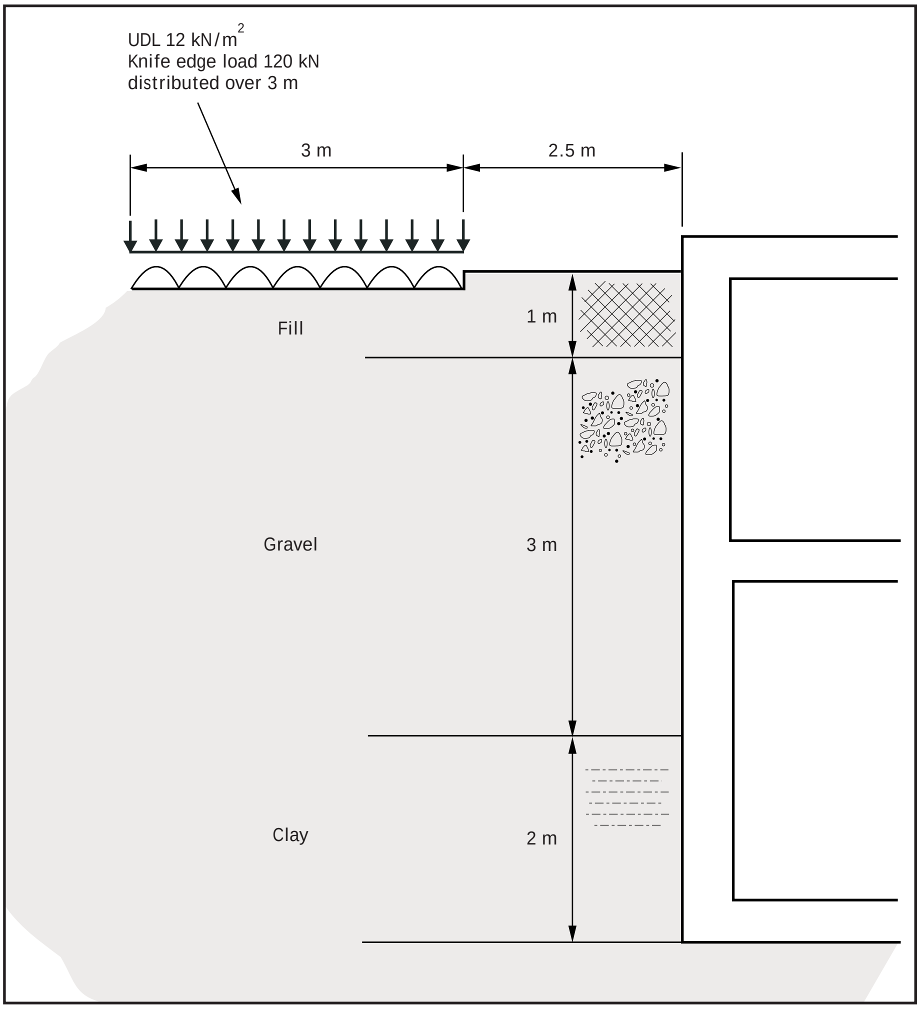
Figure 7.9 Characteristic lateral pressures – pressure at rest (kN/m^2)

7.5.3 Example: surcharge loading from imposed loads

Calculate the lateral pressure on the basement wall shown in Figure 7.7 caused by the surcharge loading of a strip load and a knife edge load indicated in Figure 7.10. Assume that the soil properties are the same as assumed in Example 7.5.1. Also assume that the basement is constructed bottom up.

Please note that the loading used in this section corresponds to HA loading, which was used for the design of bridges and, traditionally was applied to determine loads on adjacent basement structures. The equivalent might be load Model 1 to BS EN 1991-2 & UK NA [48, 48a] see Section 7.4, Imposed load: highways.

Figure 7.10
Lateral pressure due to surcharge



mpa The Concrete Centre	Project details **Calculations of surcharge loading from imposed loads**	Calculated by CG	Job no. CCIP - 044
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		Client TCC	Date

Methodology:

The uniformly distributed loading is as long as the basement. Therefore it can be treated as strip load parallel to wall using Section 7.4. The relevant parameters are shown in Figure 7.11.

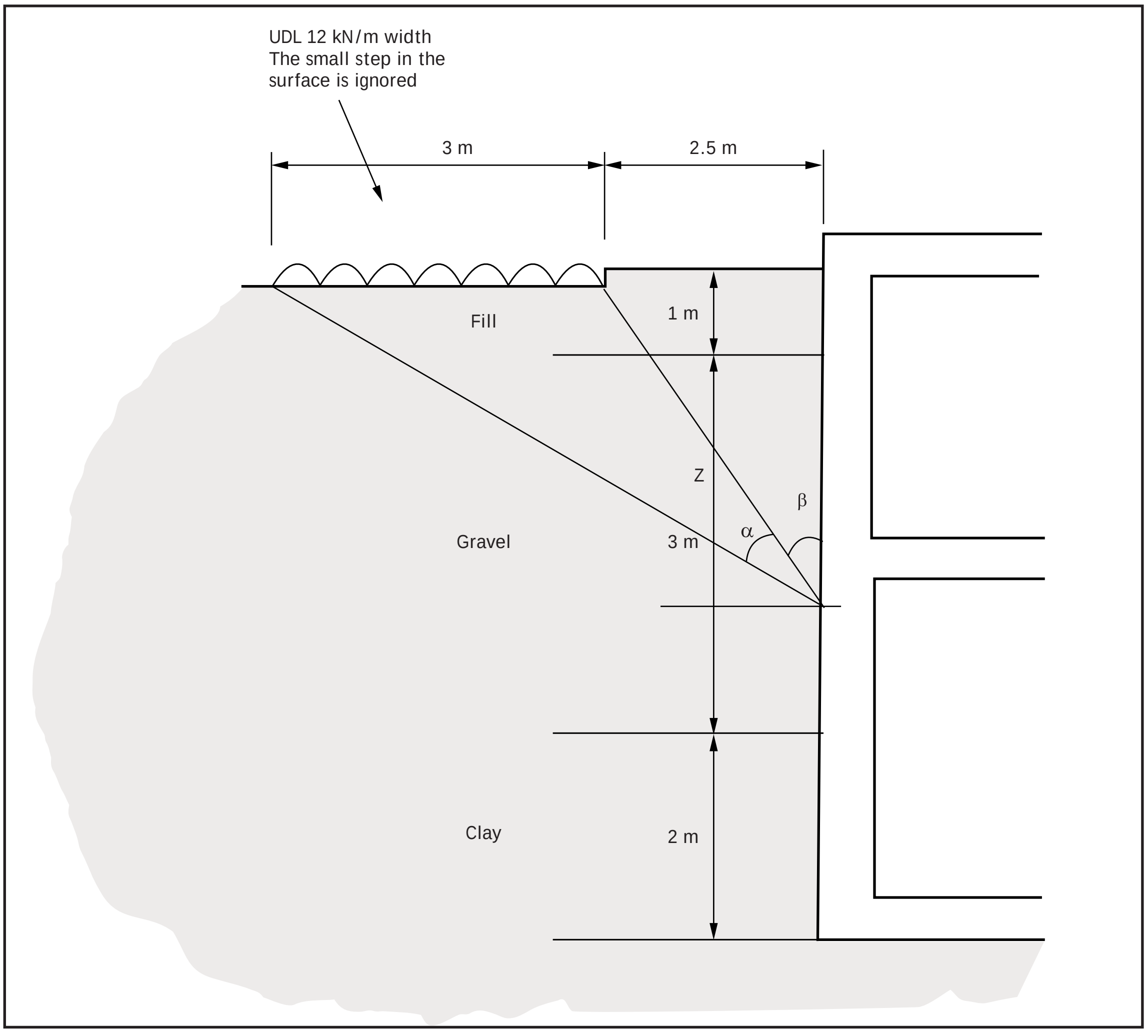


Figure 7.11 Parameters for calculation of pressure due to strip load parallel to wall.

For the strip load, a UDL surcharge, it is convenient to carry out the calculation using a tabular method.

The knife edge load is considered as equivalent to three point loads. As per Section 7.4.2, the lateral pressure is calculated for a width (i.e. length of wall) of 0.6h centred on the knife edge load. The floor-to-floor height is 3 m, so 2.0 m was chosen as the width. Pressure is calculated for (a) points in line with the knife edge load; (b) points 1.0 m from the line of the load; and (c) 2.0 m from the line of the load. The averaged pressures for structural design are obtained as shown in Table 7.9.

Strip load

As discussed in Section 7.4.4 lateral pressure caused by the 12 kN/m^2 strip load may be determined from:

$\sigma'_{ah} = K_h (q/\pi) [\alpha + \sin \alpha \cos (\alpha + 2\beta)]$ and
$q_{kh} = 2\sigma'_{ah}$

where

$K_h = K_{ad}$ or K_{Od} as appropriate. In this case (bottom up) K_{ad} values are appropriate

α, β as defined in Figure 7.11

Thus Table 7.9 can be created.

Table 7.9 Lateral pressures due to strip load for load combinations 1 and 2

1	2	3	4		5	6	7		8	9	10	11	10	11
											Combination 1		Combination 2	
Soil layer	Z (m)	Tan β = (2.5/z)	β degrees	β rads	Tan ($\alpha+\beta$) = 5.5/z	($\alpha+\beta$)	α degrees	α rads	α + sin α. cos(α + 2β)	q/π	$K_h = K_{ad}$	q_{kh} = 2 x (10) × (9) × (8) kN/m^2	$K_h = K_{ad}$	q_{kh} = 2 x (10) × (9) × (8) kN/m^2
Fill	0	∞	90.0	1.571	∞	90.0	0.0	0.0	0.0	3.82	0.333	0.0	0.410	0.0
	1	2.500	68.2	1.190	5.500	79.7	11.5	0.201	0.032	3.82	0.333	0.1	0.410	0.1
Gravel	1	2.500	68.2	1.190	5.500	79.7	11.5	0.201	0.032	3.82	0.283	0.1	0.330	0.1
	2	1.250	51.3	0.896	2.750	70.0	18.7	0.326	0.159	3.82	0.283	0.3	0.330	0.4
	3	0.833	39.8	0.695	1.833	61.4	21.6	0.377	0.305	3.82	0.283	0.7	0.330	0.8
	4	0.625	32.0	0.559	1.375	54.0	22.0	0.383	0.410	3.82	0.283	0.9	0.330	1.0
Clay	4	0.625	32.0	0.559	1.375	54.0	22.0	0.383	0.410	3.82	0.333	1.0	0.410	1.3
	5	0.500	26.6	0.464	1.100	47.7	21.2	0.369	0.467	3.82	0.333	1.2	0.410	1.5
	6	0.417	22.6	0.395	0.917	42.5	19.9	0.347	0.490	3.82	0.333	1.2	0.410	1.5

The design pressure distribution is summarised in Figure 7.13. By comparison with Tables 7.6 and 7.7 it will be seen that the lateral pressures are significantly less than those caused by the minimum 10 kN/m^2 surcharge immediately behind the wall.

Knife edge load

Knife edge load of 120 kN distributed over 3 m taken as being equivalent to three adjacent point loads of 40 kN as shown in Figure 7.12.

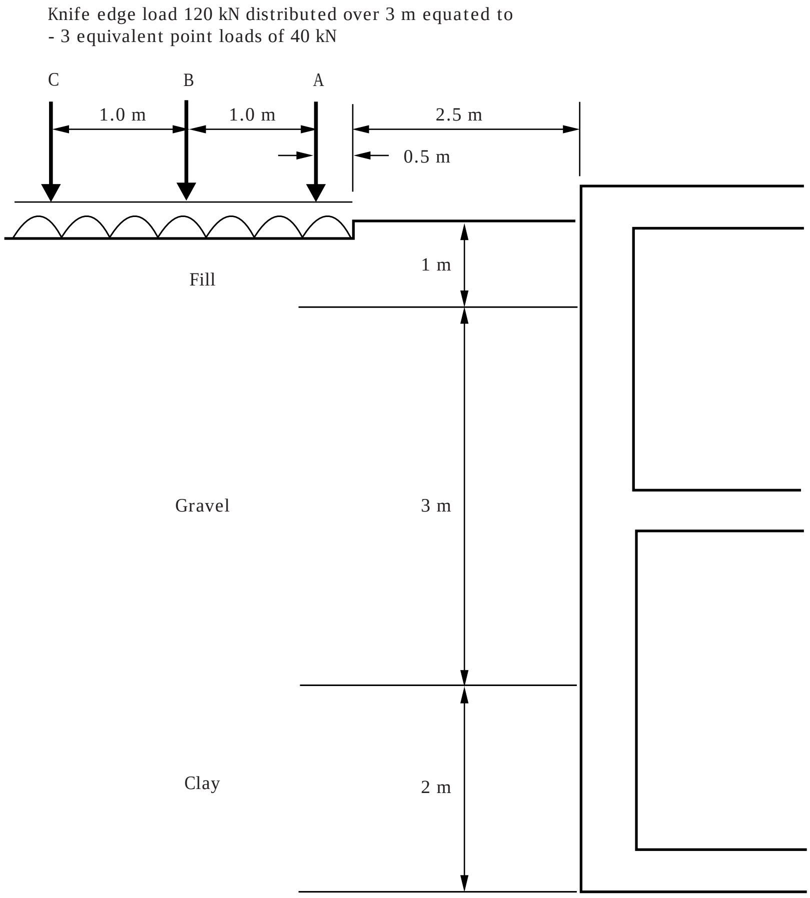


Figure 7.12 Idealisation of knife edge load

Referring to 7.4.3 and Figure 7.2 and the expression

$\sigma'_{ah} = K_h(3QZ^3)/(2\pi R^5)$.
$q_{kh} = 2\sigma'_{ah}$

where

$K_h = K_{ad}$ or K_{Od} as appropriate. In this case $= K_{ad}$
Q = load, kN = 40 kN at A, B and C
Z = depth, m
$R = (X^2 + Y^2 + Z^2)^{0.5}$

where

X = distance of load perpendicular to wall, m
Y = distance of load to point of consideration measured parallel to wall, m

The following table can be created:

Table 7.10 Lateral pressures σ'_{ah} from knife edge load considered as three point loads for Y = 0 (Combination 1) kN/m²

Soil layer	Depth	Active coefficient	Point load A for Y= 0 m, X= 3.0 m		Point load B for Y= 0 m, X= 4.0 m		Point load C for Y = 0 m, X = 5.0 m		Total pressure for Y = 0 m
	Z (m)	K_{ad}	R	σ'_{ah}	R	σ'_{ah}	R	σ'_{ah}	σ'_{ah}
Fill	0	0.333	3.00	0.00	4.00	0.00	5.00	0.00	0.00
	1	0.333	3.16	0.04	4.12	0.01	5.10	0.00	0.05
Gravel	1	0.283	3.16	0.03	4.12	0.01	5.10	0.00	0.05
	2	0.283	3.61	0.14	4.47	0.05	5.39	0.02	0.21
	3	0.283	4.24	0.21	5.00	0.09	5.83	0.04	0.35
	4	0.283	5.00	0.22	5.66	0.12	6.40	0.06	0.41
Clay	4	0.333	5.00	0.22	5.66	0.12	6.40	0.06	0.41
	5	0.333	5.83	0.24	6.40	0.15	7.07	0.09	0.47
	6	0.333	6.71	0.20	7.21	0.14	7.81	0.09	0.44

Similar tables can be created for Combination 1 and Combination 2 at Y = 0, Y = 1 m and Y = 2 m, and used to give averaged pressures. As σ'_{ah} has been determined using Boussinesq's theory of stresses, these stresses are doubled to give design pressure (i.e. $q_{kh} = 2\sigma'_{ah}$).

Table 7.11 Average design pressures, q_{kh} from knife edge load considered as three point loads (for combinations 1 and 2), kN/m²

		Combination 1 Total lateral pressures				Combination 2 Total lateral pressures			
		Y = 0 m	Y = 1 m	Y = 2 m	Average	Y = 0 m	Y = 1 m	Y = 2 m	Average
Soil layer	Z (m)	q_{kh}	q_{kh}	q_{kh}	q_{kh}	q_{kh}	q_{kh}	q_{kh}	q_{kh}
Fill	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	1	0.11	0.09	0.05	0.08	0.13	0.11	0.06	0.10
Gravel	1	0.09	0.08	0.04	0.07	0.11	0.09	0.05	0.08
	2	0.42	0.36	0.23	0.34	0.49	0.42	0.27	0.39
	3	0.70	0.62	0.45	0.59	0.81	0.72	0.53	0.69
	4	0.81	0.74	0.59	0.71	0.94	0.87	0.68	0.83
Clay	4	0.81	0.74	0.59	0.71	1.17	1.08	0.85	1.03
	5	0.95	0.89	0.74	0.86	1.17	1.09	0.91	1.06
	6	0.88	0.83	0.72	0.81	1.08	1.03	0.89	1.00

Summary

The characteristic lateral design pressure distribution from considering strip and knife edge loads are shown in Figure 7.13

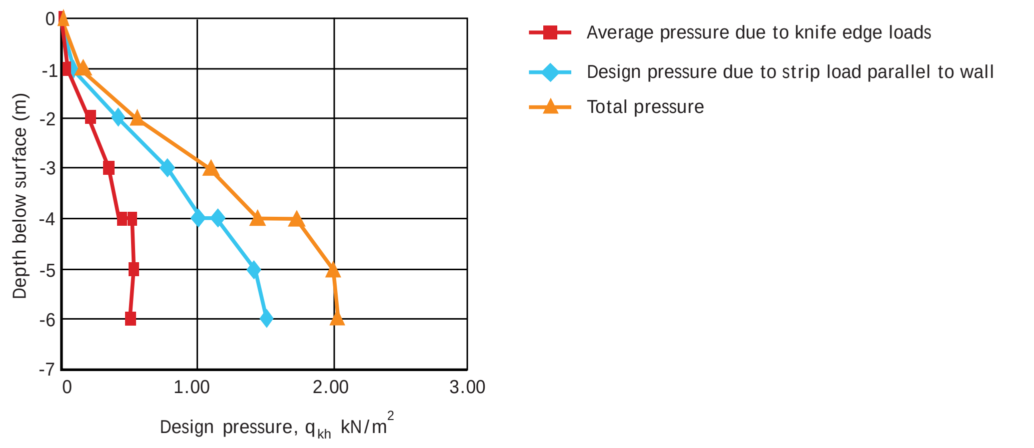


Figure 7.13 Design pressure distribution due to UDL and knife edge load (Combination 2)

7.5.4 Example: surcharge loading from a pad foundation

Calculate the lateral pressures caused by the existing pad foundation adjacent to the proposed basement in Figure 7.7 shown in Figure 7.14 and Figure 7.15. The soffit of the pad is 2 m x 2 m in plan and bears at 150 MPa at 1.5 m below EGL. Assume the soil strata and soil properties are as in Example 7.5.1.

Figure 7.14
Surcharge from a pad foundation.

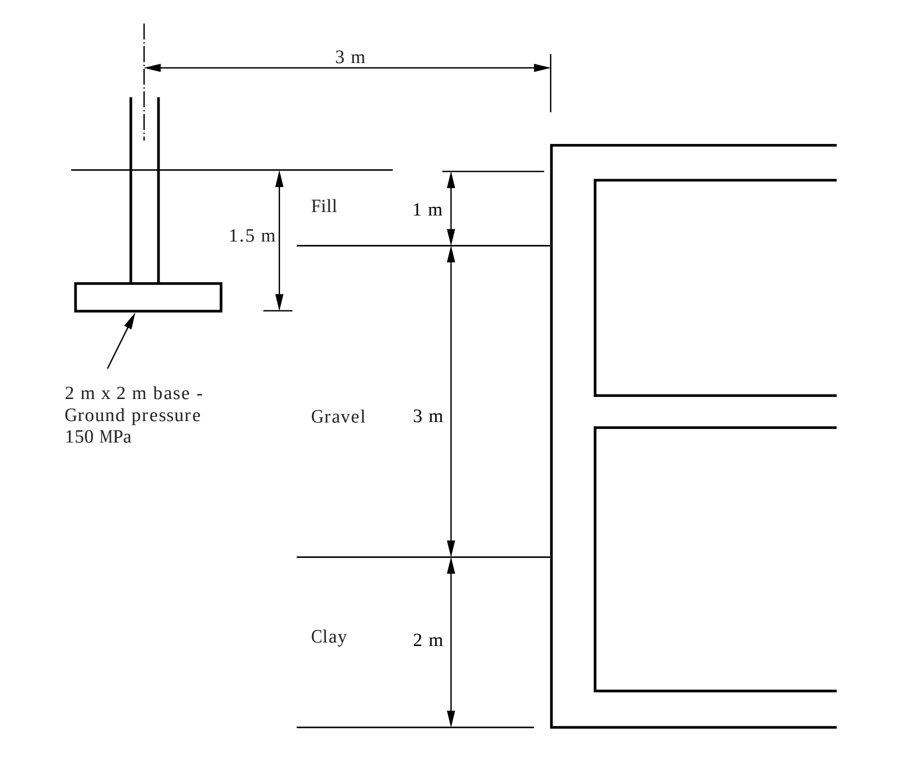
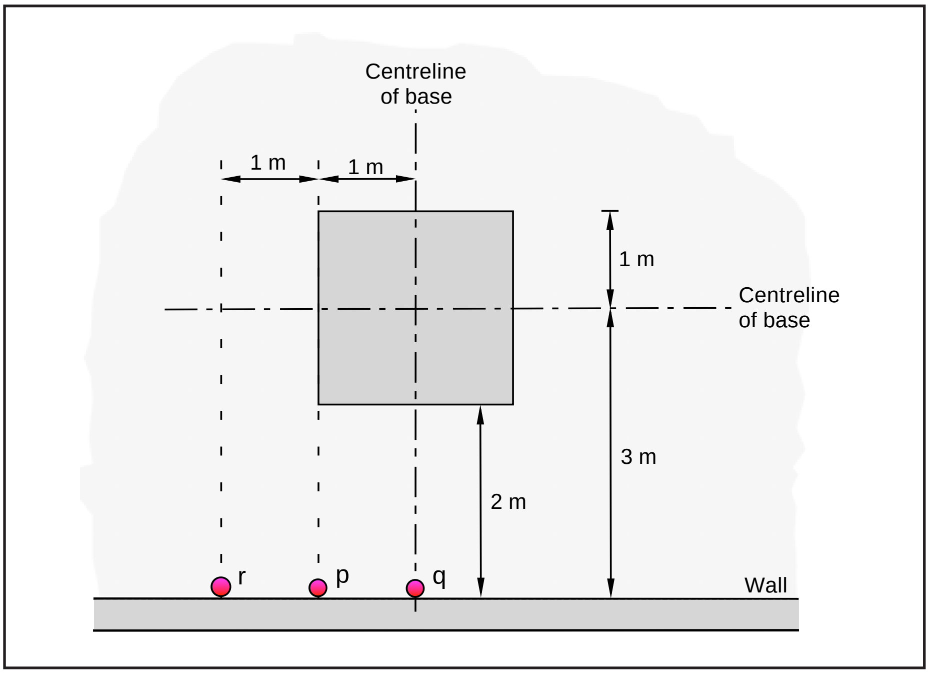


Figure 7.15
Plan of wall and foundation.



mpa The Concrete Centre	Project details: **Calculations of surcharge loading from a pad foundation**	Calculated by: CG	Job no.: CCIP - 044
		Checked by: PG	Sheet no.
		Client: TCC	Date

Using the method explained in Section 7.4.5

$\sigma_{ah} = K_h(\sigma_{v,z}) = K_{ad}\, q I_r$

where

$K_h = K_{ad}$ or K_{0d} as appropriate. In this case assume support must always be maintained. Therefore use $= K_{0d}$

$\sigma_{v,z} =$ vertical stress at depth z

$q =$ stress under rectangular plate, kN/m^2

$I_r =$ coefficient for vertical pressure under the corner of a rectangular plate dependent on the ratios of the length (m) and width of the plate (n) to the depth (z) where the pressure is required. See Figure 7.4

Superposition allows the determination of vertical stress under any point within or outside the loaded area. In turn lateral pressure can be obtained. Pressure is calculated on three lines at p, q and r as shown.

Pressures are calculated at a corner of a rectangular area viz:

Pressure at q = 2 x [(pressure at q for a rectangle 4 m x 1 m) − (pressure at q for a rectangle 2 m x 1 m)]

Pressure at p = (pressure at p for a rectangle 4 m x 2 m) − (pressure at p for a rectangle 2 m x 2 m)

Pressure at r = (pressure at r for a rectangle 4 m x 3 m) − (pressure at r for a rectangle 4 m x 1 m) − (pressure at r for a rectangle 3 m x 2 m) + (pressure at r for a rectangle 2 m x 1 m)

Table 7.12 shows the derivation of pressures on line q.

Table 7.12 Pressure on line q (Combination 2)

1	2	3	4	5	6	7	8	9	10
Depth below surface (m)	Depth below pad foundation z (m)	4 m × 1 m rectangle			2 m × 1 m rectangle			$K_h = K_{0d}$*	$\sigma_{ah} = 150 \times [(5) - (8)] \times (9)$ (kN/m²)
		m	*n*	I_r#	*m*	*n*	I_r#		
0.0								0.581	0
1.0								0.581	0
1.5	0.0	∞	∞	0.250	∞	∞	0.250	0.497	0
2.0	0.5	8.00	2.00	0.240	4.00	2.00	0.240	0.497	0
3.0	1.5	2.67	0.67	0.162	1.33	0.67	0.160	0.497	0.15
4.0	2.5	1.60	0.40	0.113	0.80	0.40	0.095	0.497	1.34
4.0	2.5	1.60	0.40	0.113	0.80	0.40	0.095	1.000	2.70
5.0	3.5	1.14	0.29	0.080	0.57	0.29	0.062	1.000	3.00
6.0	4.50	0.89	0.22	0.056	0.44	0.22	0.040	1.000	3.45

Key
From Figure 7.4
* Combination 2 values as before – see Table 7.8.

Pressures σ'_{ah} on lines p and r are calculated in a similar manner. As $q_{kn} = 2\ \sigma'_{ah}$, chacteristic design pressures would be twice the values calculated.

Figure 7.16 shows pressures calculated for lines q, p and r.

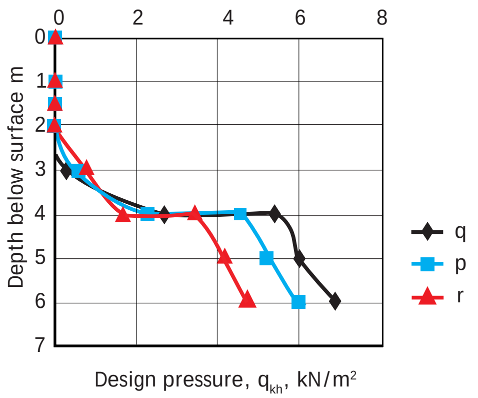


Figure 7.16 Lateral pressure q_{kh} due to surcharge from pad foundation at locations p, q and r (Combination 2)

7.5.5 Example: compaction pressures

It should be noted that during construction, the structure can be subjected to load conditions that can be more severe than those experienced in normal service. Compaction pressures are examined here.

Calculate the compaction pressures that are likely to arise behind a 4 m deep basement wall. Assume that the fill is medium dense gravel and that it is compacted using pedestrian operated 1.3 t vibrating roller.

mpa The Concrete Centre	Project details: **Calculations of ultimate compaction pressures**	Calculated by: CG	Job no.: CCIP - 044
		Checked by: PG	Sheet no.
		Client: TCC	Date

Compaction pressures are calculated for both load combinations 1 and 2 (see Section 8.2) using the relevant partial factors for the structure and soil. The partial factor for soil needs to be known to establish the pressure coefficients. Formulae in Section 7.4, Table 7.5 and Figure 7.6 are used as reference.

Property	Combination 1	Combination 2
Density of soil, γ	2225 kg/m^3	2225 kg/m^3
φ_{max} = (30° A + B + C) = 30 + 4 + 4 + 2 = 40°	40°	40°
$\tan \varphi'_d = (\tan \varphi_{max})/\gamma\varphi$	0.839/1.0 = 0.839	0.839/1.25 = 0.671
φ'_d	40°	33.8°
φ'_{crit} = (30° A + B) = 30 + 4 + 4 = 38°	38°	38°
φ'_d	38°	33.8°
For level ground $K_{pd} = (1 + \sin \varphi'_d)/(1 - \sin \varphi'_d)$	4.20	3.51
For level ground $K_{ad} = (1 - \sin \varphi'_d)/(1 + \sin \varphi'_d)$	0.238	0.285
Design force $P_d = P_K \gamma_F = 73\ \gamma_F$	73 x 1.35* = 98.6 kN	73 x 1.0 = 73 kN
$[2 P_d / \pi \gamma_{k,fill}]$	2.82	2.09
Depth of point J = $(1/K_{pd})\ [2 P_d / \pi \gamma_{k,fill}]^{0.5}$	0.40 m	0.411 m
Depth of point K = $(1/K_{ad})\ [2 P_d / \pi \gamma_{k,fill}]^{0.5}$	7.05 m (i.e. deeper than the basement)	5.07 m (i.e. deeper than the basement)
Pressure at J = $K_{pd}\ \gamma_{k,f}\ z_j$	36.7 kN/m^2	31.5 kN/m^2

Note

* γ_G assumed.

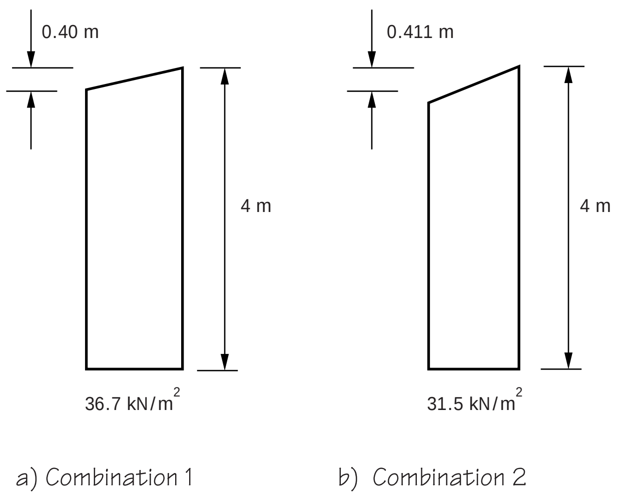


Figure 7.17 Distribution of characteristic lateral design pressure due to compaction

8. Design for ultimate limit states

Design should be carried out to comply with:

- BS EN 1990: *Basis of structural design*[6]
- BS EN 1991-1-1 *General actions – Densities, self-weight, imposed loads for buildings*[7]
- BS EN 1991-1-6 *Actions on structures during execution*[8]
- BS EN 1992-1-1 *Design of concrete structures – General rules & rules for buildings*[9]
- BS EN 1992-1-2 *Design of concrete structures – Structural fire design*[54]
- BS EN 1992-3, *Eurocode 2: Design of concrete structures, Part 3: Liquid retaining and containing structures*[10]
- BS EN 1997-1 *Geotechnical design Part 1 – General rules*[11]
- and their National Annexes
- CIRIA Report C660, *Early–age thermal crack control in concrete*[18]: this document is quoted as NCCI in the UK National Annex to BS EN 1992-3

At the ultimate limit state, equilibrium (EQU), strength of structural elements (STR) and resistance of the soil (GEO) should be verified.

Transient (construction stage) and persistent (conditions of normal use) design situations should be considered. All the loads that could occur in each design situation should be taken in to account in design. The method for combining the loads that occur simultaneously is discussed in Section 8.2.

Notional design life of the structure should be agreed with the client. This will be relevant for the selection of materials (e.g. concrete covers and composition) and definition of loads (i.e. wind loads).

8.1 Equilibrium

Verification of equilibrium (or stability) (EQU) for toppling (overturning) and sliding is not normally relevant to finished basement structures; however, both may need consideration during construction depending on the form and method of construction.

On sloping grounds failure of the ground by the rotation of soil mass (slip circle failure) will need to be considered.

Uplift (UPL) caused by buoyancy forces due to the water table should be verified at all stages during construction as well as in the completed structure. Generally floatation is unlikely to be a problem when the basement forms part of a multi-storey building, but may be critical during construction stages or where there is little or lightweight construction above.

The effects of hydraulic base heave from underlying permeable layers, internal erosion or piping in the ground due to hydraulic gradients (HYD) must also be considered.

Equilibrium should be verified in accordance with BS EN 1997-1-1. The relevant partial factors of safety for the verification are noted in Annex A of BS EN 1997-1 and are shown in Table 8.1.

Table 8.1
Partial factors for loads for equilibrium check (EQU)[6a]

Action	Stabilising – favourable	Destabilising – unfavourable
Permanent γ_G	0.9	1.1
Variable γ_Q	0.0	1.5

Note
According to Eurocode 7 and its National Annex [11a], favourable factors for UPL and HYD are similar, but unfavourable $\gamma_{G,unfav}$ = 1.1 for UPL and 1.335 for HYD.

8.2 Load combinations and partial factors

In each design situation all the actions that are likely to occur should be considered. Partial factors are applied to these actions to obtain design values. The method of arriving at the design value of actions that occur simultaneously is referred to as load combination.

Combinations 1 and 2

In any design that involves geotechnical actions (e.g. lateral earth pressures), BS EN 1990 and the National Annex to BS EN 1997-1 require consideration of two separate combinations of loads (i.e. two sets of partial factors). These partial load factors are associated with partial factors for soil properties leading to ‘Combination 1’ and ‘Combination 2’ factors at ultimate limit state (See Table 8.2). The structure and the soil are required to resist the action effects caused by both combinations.

In principle the ULS of STR and GEO are examined. Combinations 1 and 2 equate to 'combinations' or 'sets' B and C required by BS EN 1990 when considering the STR/EQU cases. Combination 1 is often said to make the structure critical and Combination 2 the soil but both combinations should be checked.

For each combination of loads, action effects should be calculated using the design values of actions obtained by applying partial factors (γ_F) to representative values and the appropriate geotechnical properties (and actions) derived by applying partial factors (γ_M) to values of soil parameters. All the loads that occur in a load case should be classified as either permanent or variable. Earth pressures should be considered as permanent actions. For water pressures, see below.

The ‘Combination 1’ and ‘Combination 2’ values of γ_F and γ_M are shown in Table 8.2. Partial factors γ_M (soil parameters) and γ_R (resistances) are used to obtain ground resistance values. The National Annex[11a] should be consulted for values of γ_R.

As noted earlier, lateral earth pressures on retaining structures is obtained using design values of the effective angle of shearing resistance φ_d'. However, the value of φ_d' depends on the partial factor for the soil parameter γ_M and so for the two combinations to be considered γ_φ varies. Thus the partial factor for material governs the design value of the load. In design calculations values of φ_d' should be established for each combination.

Table 8.2
Load combinations and partial factors (ULS).

Combination	Partial factors on actions, γ_F			Partial factors on soil properties, γ_M			
	$\gamma_{G,unfav}$	$\gamma_{G,fav}$	γ_Q	γ_φ #	$\gamma_{c'}$	γ_{cu}	γ_γ
1	1.35*^	1.00	1.50	1.00	1.00	1.00	1.00
2	1.00	1.00	1.30	1.25	1.25	1.40	1.00

Note
Values for piles not shown: refer to BS EN 1997-1 Annex A.
There are three design approaches in Eurocode 7[11]. The UK National Annex[11a] adopts design approach 1 (DA1). DA1 requires the consideration of two combinations of partial factors for both actions and soil parameters in order to compare ultimate loads with ultimate soil resistance. The two combinations are detailed in BS EN 1990 Annex A: Tables A1.2(B) & A1.2(C).

Key
γ_φ is applied to tan φ.
* Eurocode 7 treats groundwater as a permanent action. However, as detailed below, it is recommended that a $\gamma_{G,unfav}$ factor of 1.35 is applied to water pressure from water in 'normal' conditions and in a separate verification, a $\gamma_{Q,unfav}$ factor of 1.20 (or 0) is applied to pressure from water at the most unfavourable level that could occur during the lifetime of the structure (i.e. usually, at surface level).
^ For compaction pressures, partial factor γ_F is presumed = γ_G.

γ_F for ground water

Eurocode 7 treats ground water as a permanent action, therefore, at 'normal' water levels and using Combination 1, $\gamma_F = \gamma_G = 1.35$. However, γ_F for 'abnormal' water levels bear some discussion.

With respect to the water pressure derived on the basis of design water table at ground surface, it is tempting to suggest a value of 1.0 should be applied (e.g. using Combination 2). However, this may not be strictly sufficient for design of the structure. BS EN 1990 states that the purpose of the partial factor γ_F is to:

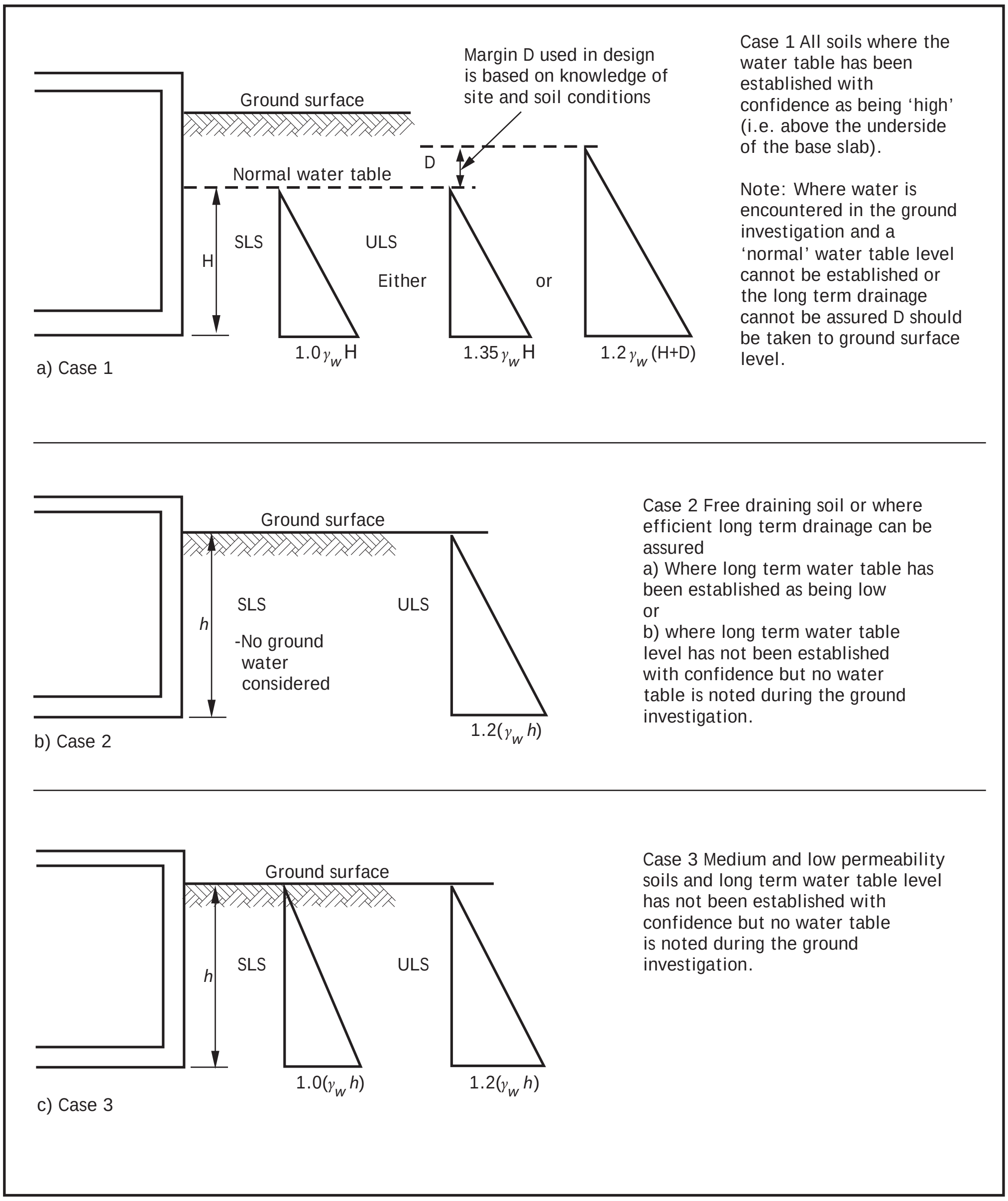
a) account for the unfavourable deviation of the action values from the representative values;
b) the modelling effects of actions; and
c) in some cases the modelling of actions.

Other reasons usually accepted, although not stated in BS EN 1990, are unforeseen redistribution of stresses and variations of geometry of the structure or its elements, as this affects the determination of the action effects. Using a design water table at ground surface only deals with purpose a) noted above. Therefore a value greater than 1.0 is required after uncertainties of the value of actions have been addressed. In Table 2.1 BS 8110-1:1997 gives some guidance. It recommends a value of 1.2 when the maximum credible level for the water can be clearly defined; otherwise a value of 1.4 is recommended. Thus when uncertainties in the value actions are removed, the normal load factor is reduced by (1.4/1.2) = 1.17. If the same margin is applied to the γ_F normally applied to permanent actions, the reduced value would be (1.35/1.17) = 1.15.

This figure is in line with the UK NA[55a] to BS EN 1991-4 Silos and tanks[55] which states that for the liquid induced actions γ_Q may be taken to equal 1.20 during operation, applied to the stored liquid at the maximum design liquid level.

Thus, on the above basis, it is recommended that for ULS verification $\gamma_F = \gamma_{G,unfav} = 1.35$ should be applied to 'normal' ground water levels and $\gamma_F = \gamma_Q = 1.20$ should be applied to pressure from water at the most unfavourable level that could occur during the lifetime of the structure (i.e. at surface level or, where the ground water level is known with confidence, at the ground water level plus a margin based on knowledge of the site and soil conditions). The ground water levels and appropriate partial factors are illustrated in Figure 8.1.

Figure 8.1
Ground water levels and partial factors for ground water to be used in design.



8.3 Verification of structural elements

Verification should be carried out using BS EN 1992-1-1 and BS EN 1992-1-2. Analysis of a vertical section is usually undertaken: the form of the actions and resulting bending moment diagram for a typical propped wall situation is shown in Figure 8.2. Analysis should, however, also take account of continuity with suspended floor slabs and the possible three-dimensional nature of basement structures, including interactions between wall and base slab elements.

When the aspect ratio (wall length/ wall height) is less than about 1.5, external pressures on walls may be resisted by a combination of horizontal and vertical moments. Appendix A4 provides charts for design in the vertical and horizontal directions.

Moment transfer between walls at corners should be considered and the walls should be suitably reinforced. Figures 8.2 and 8.3 illustrate some typical cases.

It is normal practice first to design the completed structure and then verify it for the action effects during different stages of construction, based on the assumed sequence of construction and stipulated propping requirements. If the contractor wishes to change the method of construction the design would normally be reappraised by the contractor and modified to the satisfaction of the designer.

When considering shear, care should be exercised in taking advantage of any axial compression caused by earth pressures. It will be safe to ignore it. *Concise Eurocode 2*[56] and the compendium, *How to design concrete structures using Eurocode 2*[57] provide useful guidance on the ULS design of concrete elements.

Figure 8.2
Vertical section through a basement showing typical actions on and moments in walls and slab.

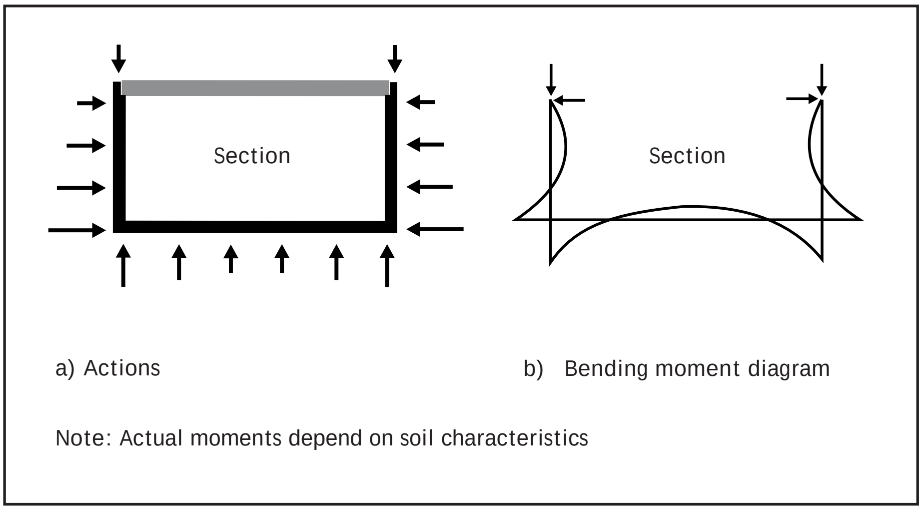
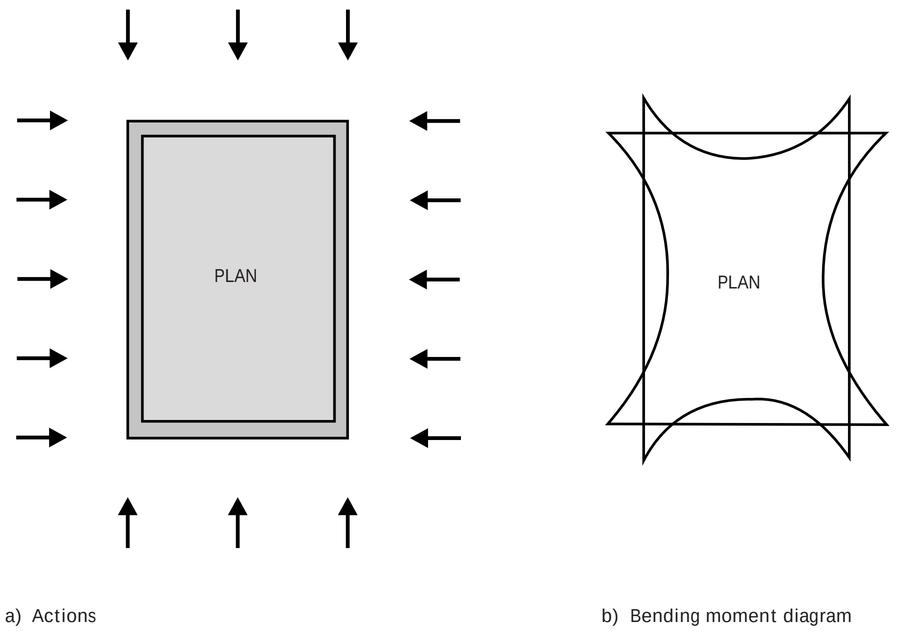


Figure 8.3
Plan section through a basement showing typical actions on and moments in walls.



Note: Horizontal bending will not usually be the controlling action effect unless the aspect ratio of the wall i.e. height/width is greater than about 0.66.

9. Design for serviceability limit states

The Serviceability Limit State of cracking will often govern the amount of reinforcement provided in basement structures. In particular, it is necessary to verify the control of cracking due to restraint to early thermal and longer term shrinkage movements.

For design at the SLS, it is generally sufficient to verify deflections to BS EN 1992-1-1[9, 9a] and more critically, cracking to BS EN 1992-3[10, 10a] or CIRIA C660[18].

Actions and partial factors

As discussed in Section 7.2, the lateral earth pressure will be at its highest value at serviceability limit state since the deformations will be small. Earth pressures should be derived using Section 7.3. Where it is not possible to determine the 'normal' water table with confidence, it should be assumed to be at the ground surface level.

According to BS EN 1997-1[11] Clause 2.4.8(2) (and the UK NA[11a]) the partial factors for serviceability limit states are normally taken as 1.00. See Table 9.1.

Table 9.1
Partial factors for actions at serviceability limit state.

	Partial factors on actions			Partial factors on soil properties			
	$\gamma_{G,unfav}$	$\gamma_{G,fav}$	γ_Q	γ_φ	$\gamma_{c'}$	γ_{cu}	γ_γ
Serviceability combination	1.00	1.00	1.00	1.00	1.00	1.00	1.00

9.1 Causes of cracking and general principles of crack control

Before considering the process to be used for estimating crack widths, it is worth noting the various conditions for which crack widths are to be calculated. For common cases, these are as follows:

1. Cracking caused by restraint to movement (also referred to as imposed deformations). Examples include:
   - early thermal effects
   - autogenous shrinkage
   - drying shrinkage
2. Cracking caused by loading:
   - flexure
   - axial tension

For cracking to occur part or all of the concrete section has to be in tension. Initial cracking is usually caused by early age thermal effects, and these cracks will usually pass right through the element. In the long term restraint to movement due to drying shrinkage (including autogenous shrinkage) may be sufficient to crack the concrete (see Section 9.1.4).

Cracking is likely and assumed to occur when the restrained strain, ε_r, exceeds the tensile strain capacity of the concrete ε_{ctu}. (See Section 9.4)

The tensile force in the concrete just prior to the first crack should be transmitted by the reinforcement without yielding to achieve controlled cracking: i.e. a minimum amount of reinforcement is provided so that small cracks occur at intervals rather than there being one large crack (see Section 9.5). This minimum amount of reinforcement may not be sufficient to restrict the crack to a particular width. The widths of these 'small cracks' are further controlled to limits set out in Section 9.6 by providing sufficient reinforcement. Crack width is predicted by multiplying crack inducing strain, (the strain dissipated by the occurrence of cracking) ε_{cr}, by crack spacing, $s_{r,max}$, according to the formulae presented in Section 9.7.

In calculating crack widths it must be remembered that concrete is an inherently variable material. The risk of cracking is influenced by many factors which are all subject to both variability and uncertainty. Consequently, crack width calculations based on single values of key parameters (set to provide conservative solutions) can only be considered as estimates. The probabilities of calculated crack widths being exceeded cannot easily be established. **However, it is assumed that the target limiting crack widths will give satisfactory performance.**

Some salient background is given below.

9.1.1 Early thermal effects

Heat is released during hydration of cement in concrete. A peak temperature above the ambient is reached relatively soon after concreting. As the rate of heat generation reduces progressively, heat loss becomes dominant and the concrete starts to cool down to ambient temperature and contracts.

Restraint to this contraction leads to tensile stresses which could lead to cracking. Restraint might be either internal to the element or external to it.

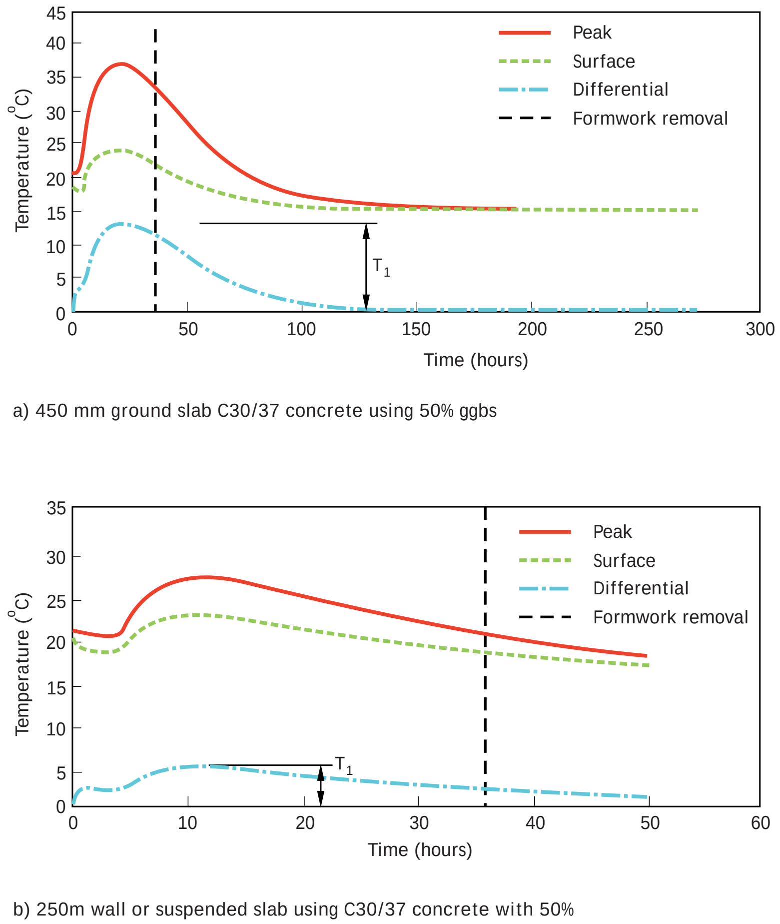
In thin sections early thermal cracking occurs within a few days of casting the concrete. It may take longer in thick sections, which cool more slowly. Figures 9.1a and 9.1b show respectively the likely temperature profiles for a 450 mm ground slab and 250 mm wall (or a suspended slab). The temperature drop (the difference between peak and ambient temperature, T_1) is used in crack width calculation. The important influence of cement type upon hydration rate and hence temperature should be recognised. For a slab cast on the ground, the ground provides insulation on one face and CIRIA C660 bases its calculation of T_1 on 1.30 × actual depth. Figure 9.2 illustrates the effects of different variables on temperature drop after cooling to ambient conditions. Table A5 shows typical values of T_1 for different C30/37 concretes.

Stresses that develop during the early thermal cycle are difficult to predict. There is a rapid change of elastic modulus. Creep causes relaxation of stresses. However, there is also a reduction in tensile strength at which failure occurs under sustained load.

Simplifying assumptions are made and the general model used is of the form:

Restrained strain = (free strain) × (modifier for creep effects) × (restraint factor).

Figure 9.1
Early age temperature profiles.

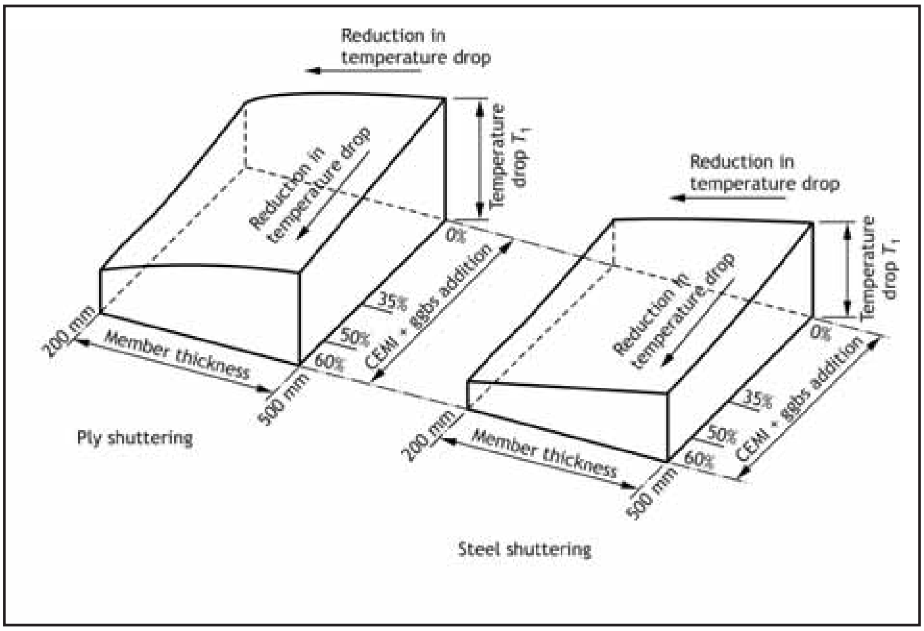


Notes

1 Figure produced using temperature differential calculation in CIRIA C660[18]

2 T_1 indicated here assumes that concrete surface is at ambient temperature

Figure 9.2
Early thermal effects – effect of different variables on temperature drop.



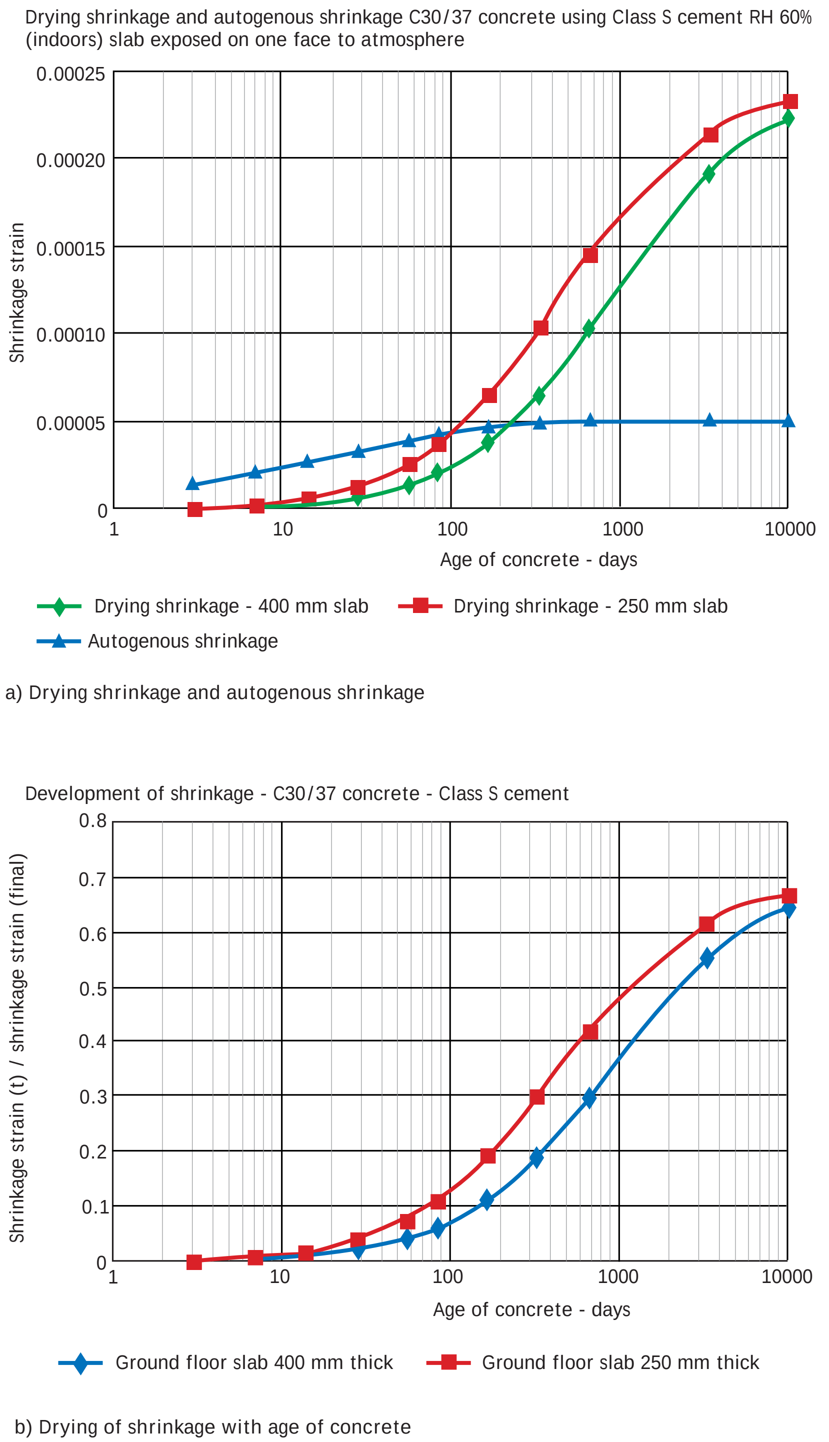
9.1.2 Autogenous and drying shrinkage

There are two types of shrinkage that need consideration in design. Autogenous shrinkage occurs during the early months after setting of the concrete. It happens in practice in the interior of a concrete mass. The contraction of the cement paste is restrained by the rigid skeleton of the already hydrated cement paste and by the aggregate particles. According to BS EN 1992-1-1 autogenous shrinkage occurs in all concretes and is directly related to the concrete strength.

Drying shrinkage is the result of volume change that accompanies the loss of moisture over months and years from the concrete to the atmosphere. Thus the relative humidity (RH) of the environment, the amount and nature of the cementitious binder (cement Class) and the member size play important roles. Portland cement rich concretes will shrink more than leaner concretes. All other things being equal, shrinkage will decrease as cement Class changes from R to N to S. In the UK, indoor concrete will shrink more readily compared with external concrete, as the atmosphere will be drier. Similarly, as moisture loss will be easier in thin sections compared with thick members they will shrink more quickly.

Shrinkage strains develop over a long period (decades: See BS EN 1991-1-1 Exp. (3.9)). Figure 9.3a shows the relative magnitudes of drying and autogenous shrinkage for a 450 mm and a 250 mm ground slab. Figure 9.3b indicates the proportion of the final shrinkage that occurs at different ages of concrete. It will be seen that only about 70% of the final shrinkage will have occurred after 10 000 days. The figures illustrate the general trends discussed above. It will be seen that autogenous shrinkage strain is an order of magnitude smaller than the long-term shrinkage strains (see also Tables A9 and A10).

Figure 9.3
Development of shrinkage with age.



Note
Based on BS EN 1992-1-1 for a 400 mm and a 250 mm thick ground slab, assuming C30/37 concrete, Class S cement and 60% RH, one face exposed to atmosphere

For slabs cast on the ground, consideration should also be given to the possibilities of cracking caused by differential drying shrinkage between the surface exposed to the air and that in contact with the ground. The gradient at the strains implies a curvature $(1/r) = \varepsilon_{outside} - \varepsilon_{outside}/h)$ and a moment of EI (1/r) to be accounted for.

Exposed concrete

Clause 7.4 of BS 8110-2[58] notes: Concrete exposed to the outdoor climate in the UK will exhibit seasonal cyclic strains of 0.4 times the 30 year shrinkage superimposed on the average shrinkage strain; the maximum shrinkage will occur at the end of each summer. No similar guidance exists in BS EN 1992-1-1. As the seasonal temperature effects are usually greater (and allowed for in design), the seasonal variation of shrinkage can generally be ignored[59].

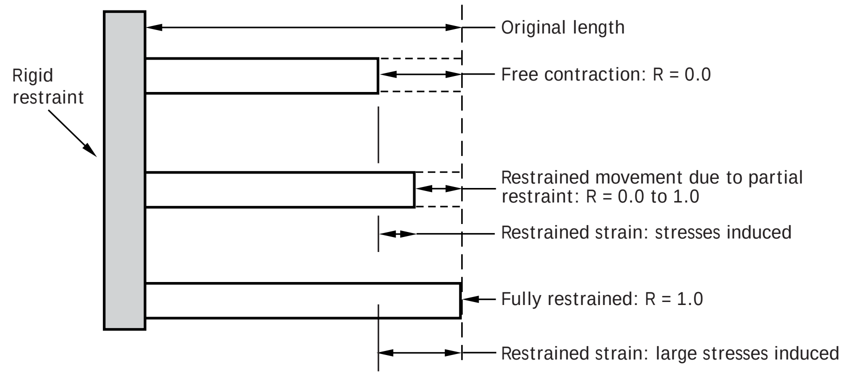
9.1.3 Restraints

Tensile forces and cracks will only develop if movement is restrained. See the schematic model for end restraint shown in Figure 9.4.

Restraint may be external or internal to the element. External restraint takes two forms: restraint at *ends* and restraint along one or more *edges*. The mechanism of crack formation in these two cases is assumed to be different and this is reflected in the formulae used for the calculation of crack inducing strain.

Internal restraint will be significant generally in thick sections (about 1 m or more), in which differential expansion causes internal restraint. Internal restraint is not considered further in this publication. Where thick slabs are used reference should be made to CIRIA C660[18].

Figure 9.4
Effect of end restraint.



Note:
A restraint factor, R, is used in the calculation of crack inducing strain. Many charts (e.g. Figure L.1 in BS EN 1992-3 include the effect of creep: thus 'fully restrained' appears as a factor of 0.5.

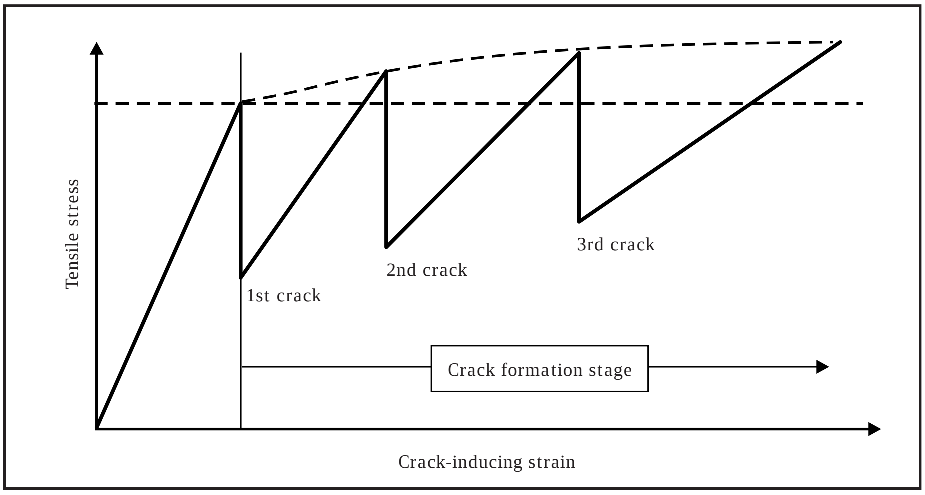
End restraint

End restraint typically occurs in:

- infill bays
- ground slab cast on piles
- large area ground slabs, restrained locally, e.g. by piles, columns or column foundations, or by a build up of friction
- suspended slab cast between rigid cores, walls or columns
- walls cast against secant, contiguous concrete or steel sheet piled walls

In the case of members restrained at ends, e*ach crack occurs to its full potential width* before a successive crack occurs (see Figure 9.5). In this case crack-inducing strain is specifically related to the strength of the concrete and the steel ratio.

Figure 9.5
Crack formation in elements restrained at ends.



Edge restraint

The case of edge restraint, for example adjacent slab pours or a wall poured onto an existing base as illustrated in Figure 9.6, differs significantly from the condition of end restraint. The principal difference is that, along with the steel, the adjacent concrete also acts as a crack distributor. Formation of a crack in this case only influences the distribution of stresses locally and the crack width is a function of the restrained strain rather than the tensile capacity of the concrete.

9.1.4 Cracking due to restraint (early thermal and shrinkage effects)

The cumulative effect of the different contractions is illustrated in Figure 9.7. This is drawn to illustrate the effects on a basement ground slab. Cracking would occur if line 2 (stress) were to cross line 1 (strength).

It is important to note that there is a critical early period of three to ten days while the concrete is still immature. After a 'honeymoon' period of months or (more usually) years, the risk becomes more critical again as the concrete become subject to drying shrinkage and by that time is more mature and stronger. Actual tensile stresses and

strengths at any point can only be estimated. Seasonal variations in temperature and drying shrinkage should also be considered and these will increase the risk of cracking. It is therefore usual to be cautious.

Figure 9.6
Edge restraint on wall cast on base (or slab cast against previously cast slab).

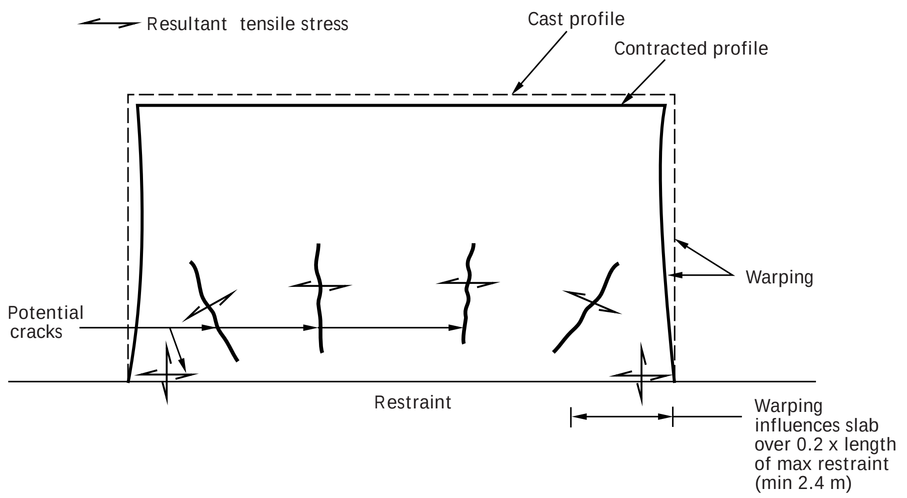
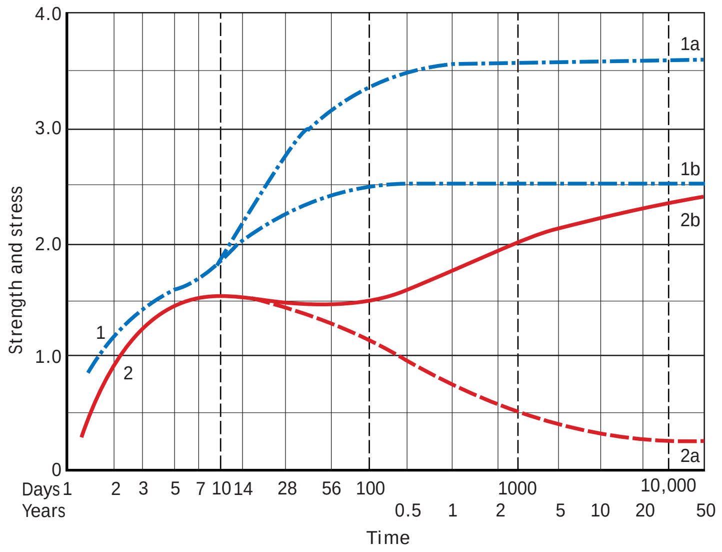


Figure 9.7
Tensile strength and stress over time[59].



Key
Line 1a Tensile strength of the concrete: transitory loading
Line 1b Tensile strength of the concrete: sustained loading
Line 2a Early-age contraction stress, allowing for creep
Line 2b Addition of drying shrinkage stress to line 2a

Note
Seasonal effects not shown

If the concrete is in a benign environment (such as a water storage tank, often with a covering of soil on the roof), the stresses due to early-age effects (moderated by creep relief) and minimal drying shrinkage may never be enough to exceed the tensile strength of the concrete and so may not crack. This is one reason why it may be inappropriate to rely on guidance for water-retaining concrete in the design of basements.

In basements, the internal faces are open to the atmosphere and therefore drying shrinkage will occur. Temperature variations may also need to be considered. Basement car parks, especially if they are naturally ventilated, are particularly susceptible as the concrete is exposed to near-ambient temperature and humidity all year round.

As explained later, it is usually more convenient to undertake calculations in terms of strain rather than stress.

9.1.5 Cracking due to flexure

Flexural cracking occurs when the tensile stress (or strain) in extreme fibres exceeds the tensile strength (or strain capacity) of the concrete. It is controlled by providing reinforcement. All codes of practice prescribe minimum reinforcement in flexural members. Such reinforcement is intended to prevent brittle failure but may be insufficient to control cracking due to tension due to the effects of restraint (as described above and in Section 9.2).

9.1.6 Cracking due to combinations of restraint and loading

Initial cracking may occur due to a combination of flexural and tension stresses (or strains) caused by loading and the effects of restraint. Generally, crack inducing strain due to restraint will be much larger than that due to flexure. Generally, restraint cracks will be through cracks, flexural cracks will only penetrate part of the thickness.

It should be pointed out that cracking in basement walls caused by restraint to movement is predominantly vertical whereas the cracking caused by loading is in many cases, depending on the aspect ratio of the wall, horizontal. This is illustrated in Figures 9.8 and 9.9. It will be seen that, whilst they should be considered, the combined effects of restraint and loading are rarely critical in the walls of basements of modest depth.

However, in basement slabs the effects of loading and restraint are likely to coincide. Whilst the effects of restraint are likely to overwhelm those from loading, design should consider the effects of restraint and flexure.

Figure 9.8
Schematic illustration of cracking caused by restraint.

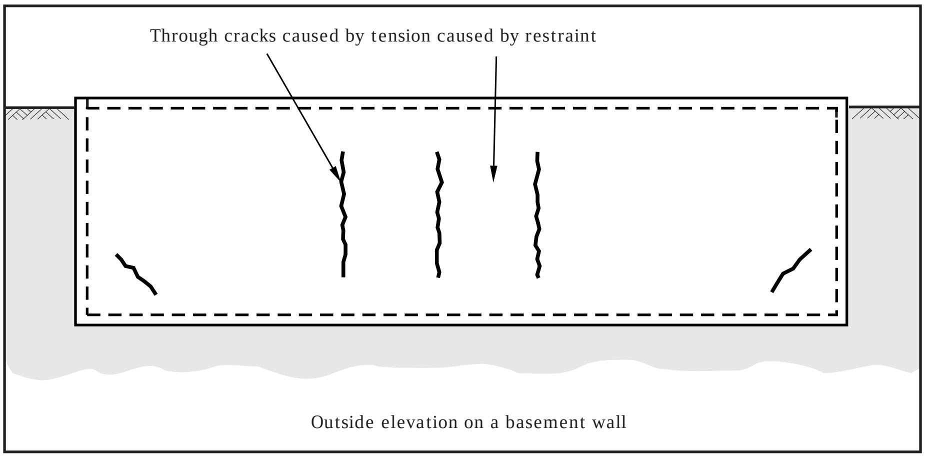
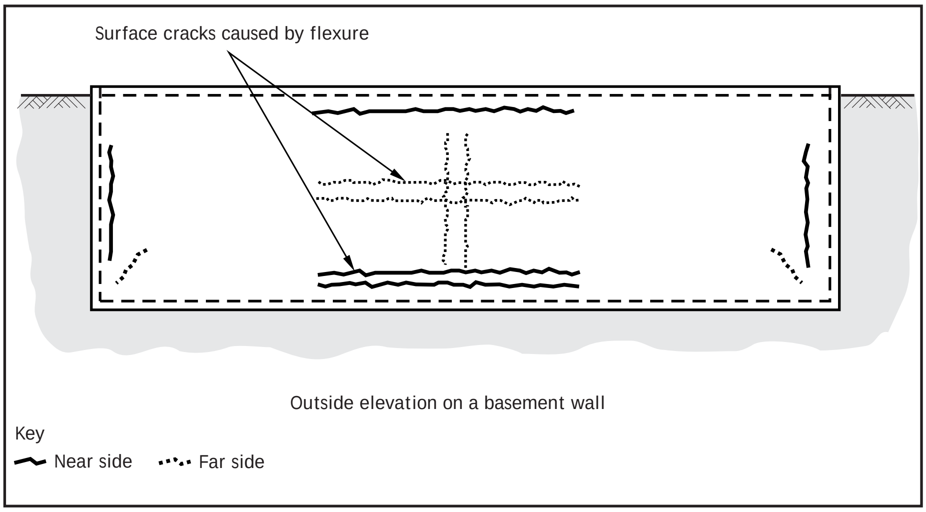


Figure 9.9
Schematic illustration of cracking caused by lateral loading.



9.2 General principles of crack control and minimum reinforcement

While cracking is accepted in concrete structures, it is expected to be controlled such that cracks occur at intervals and their width will be small. This requires the presence of a minimum amount of reinforcement in the structural element (see Section 9.5). The amount is calculated using the following philosophy.

Consider a reinforced concrete member subject to in-plane tension caused by restrained strain. Just prior to the occurrence of the first through crack the concrete and the reinforcement will be in tension. At the crack the tension can be carried only by the reinforcement. If the reinforcement is light, such that the stress in the reinforcement exceeds its yield strength, any further loading will simply result in uncontrolled widening of the first crack. If the reinforcement is more substantial such that it does not yield, application of any further strain will cause the next crack to form at the next location of weak concrete. By similar argument it can be seen that by providing sufficient reinforcement, the structure has an opportunity to develop a number of cracks. The minimum reinforcement is calculated such that the tensile strength of the reinforcement equals the tensile strength of concrete. It follows

therefore that stronger the concrete the greater will be the reinforcement required to achieve controlled cracking.

Designers should be alert to the real possibility that the strength of concrete supplied on site could be higher than specified. According to BS EN 1992-1-1, the mean strength of concrete is $f_{ck} + 8$. Producers use this figure as a target mean strength but there may be situations where a larger margin may be used. In critical applications it may be necessary to specify a *maximum* as well as a *minimum strength* for concrete. The appropriate value that should be used in the calculation of minimum reinforcement is the tensile strength of concrete at the time when first cracking might be expected to occur. Generally this will be the three-day strength when cracking due to early thermal contraction might occur, although longer term values may be more relevant if shrinkage dominates.

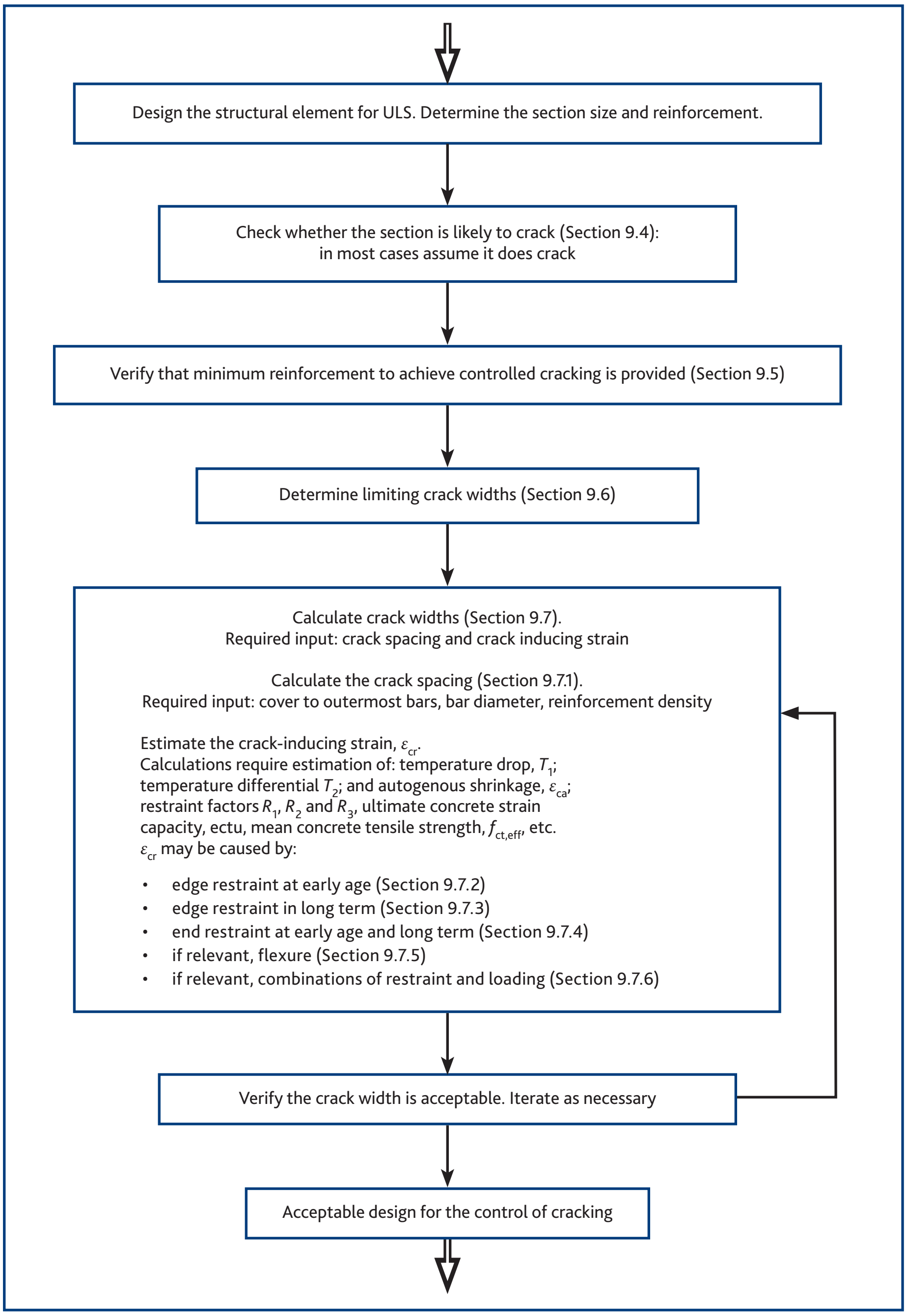
Provision of this minimum reinforcement does not guarantee any specific crack width. It is simply a necessary amount presumed by models to control cracking; but not necessarily a sufficient amount to limit actual crack widths. Additional reinforcement may well be necessary as detailed in Section 9.7 and subsequent sections.

9.3 Sequence for verification of cracking

Verification should be carried out both for early age and long term effects. Early thermal effects and autogenous shrinkage will dominate at early age and annual temperature changes, drying shrinkage and load effects will be the primary causes of cracking in the longer term.

The normal sequence of verification of cracking due to restrained deformation is shown in Figure 9.10.

Figure 9.10
Suggested procedure for verifying crack control.



9.4 Test for restraint cracking

Cracking occurs in concrete when the tensile strain builds up to exceed the tensile strain capacity of the concrete. So the restrained strain (ε_r) should be compared with the tensile strain capacity of the concrete (ε_{ctu}) to check whether the section will crack. The section will crack if:

$$\varepsilon_r > \varepsilon_{ctu}$$

CIRIA C660 3.2

where

ε_{ctu} = tensile strain capacity may be obtained from Eurocode 2 or CIRIA C660 for both short term and long term values (see Table A14).

$$\varepsilon_r = R_{ax}\,\varepsilon_{free},$$
$$= K[(\alpha_c T_1 + \varepsilon_{ca})\,R_1 + (\alpha_c T_2\,R_2) + \varepsilon_{cd}\,R_3]$$

where

R_{ax}	=	restraint factor
ε_{free}	=	strain that would occur if the member was completely unrestrained
K	=	allowance for creep
	=	0.65 when R is calculated using CIRIA C660
	=	1.0 when R is calculated using BS EN 1992-3
α_c	=	coefficient of thermal expansion (See CIRIA C660 for values). See Table A6 for typical values
T_1	=	difference between the peak temperature of concrete during hydration and ambient temperature °C (See CIRIA C660). Typical values are noted in Table A7
ε_{ca}	=	Autogenous shrinkage strain – value for early age (3 days: see Table A9)
R_1, R_2, R_3	=	restraint factors. See Section A5.6 For edge restraint from Figure L1 of BS EN 1992-3 for short- and long-term thermal and long-term drying situations. For base-wall restraint they may be calculated in accordance with CIRIA C660. Figure L1 may be used with CIRIA C660 methods providing an adjustment for creep is made (see Figure A2 and note). For end restraint, where the restraint is truly rigid 1.0 is most often used, for instance in infill bays. This figure might be overly pessimistic for piled slabs.
T_2	=	long-term drop in temperature after concreting, °C. T_2 depends on the ambient temperature during concreting. The recommended values from CIRIA C660 for T_2 are 20°C for concrete cast in the summer and 10°C for concrete cast in winter. These figures are based on HA BD 28/87[60] based on monthly air temperatures for exposed bridges. Basements are likely to follow soil temperatures so T_2 = 12°C may be considered appropriate at depth.
ε_{cd}	=	drying shrinkage strain, dependent on ambient RH, cement content and member size (see BS EN 1992-1-1 Exp. (3.9) or CIRIA C660 or Table A10). CIRIA C660 alludes to 45% RH for internal conditions and 85% for external conditions.

The risk of cracking should be assessed for both early age and long term conditions. However, the likelihood of cracking occurring is difficult to predict, and the preferred strategy is to assume that cracking will occur and to provide enough reinforcement to control it. **Thus this publication is generally based on the premise that concrete will crack and the cracking should be controlled by suitable amounts of reinforcement**.

After cracking, the residual restrained strain in the concrete is assumed to average half the tensile strain capacity [18].

9.5 Minimum reinforcement

Controlled cracking can only be achieved by ensuring that the reinforcement does not yield by ensuring that a certain minimum amount of reinforcement is provided. The minimum area of reinforcement required in BS EN 1992-1-1 is:

$$A_{s,min} = k_c\, k\, A_{ct}\, (f_{ct,eff}/f_{yk})$$

BS EN 1992-1-1 Exp. 7.1

where

k_c = A coefficient to account for stress distribution.
= 1.0 for pure tension (0.4 for pure bending).
When cracking first occurs the cause is usually early thermal effects and the whole section is likely to be in tension.

k = A coefficient to account for self-equilibrating stresses
= 1.0 for thickness h < 300 mm and 0.65 for h > 800 mm (interpolation allowed for thicknesses between 300 mm and 800 mm).

A_{ct} = area of concrete in the tension zone just prior to onset of cracking.
A_{ct} is determined from section properties but generally for basement slabs and walls is most often based on full thickness of the section.

$f_{ct,eff}$ = f_{ctm}
= mean tensile strength when cracking may be first expected to occur:
- for early thermal effects 3 days
- for long-term effects, 28 days (which considered to be a reasonable approximation)

See Table A5 for typical values.

CIRIA C660[18] and recent research[61] would suggest that a factor of 0.8 should be applied to $f_{ct,eff}$ in the formula for crack inducing strain due to end restraint and might be used here. This factor accounts for long-term loading, in-situ strengths compared with laboratory strengths and the fact that the concrete will crack at its weakest point. TR 59[62] concludes that the tensile strength of concrete subjected to sustained tensile stress reduces with time to 60–70% of its instantaneous value.

f_{yk} = characteristic yield strength of the reinforcement.
= 500 MPa

The area of reinforcement obtained using this expression may well need increasing during the design process to limit crack widths. Whilst the check should be carried out using both 3 and 28 day values of $f_{ct,eff}$ the latter will normally dominate, due to considerations of drying shrinkage, see Section 9.1.4.

9.6 Crack widths and watertightness

9.6.1 Tightness Classes

Codes of Practice prescribe limiting crack widths to achieve watertightness in *water-containing* structures. The Tightness Classes in BS EN 1992-3[10] are summarised in Table 9.2 and Notes A to D that follow.

Some engineering judgment is required in extrapolating this table to *water-excluding* structures. This judgment should be based on the analysis of risks for the site and structure under consideration in line with establishing the grade and type of basement required. A major consideration is whether the water table is likely to be low, variable or permanently high. It will be recalled from Section 3.3, that only Type B construction is required to be watertight without reliance upon applied waterproofing or cavity drainage.

Table 9.2
Tightness Classes in BS EN 1992-3

Tightness Class	Requirements for leakage	Suggested measures to meet the requirements
0	Leakage of liquids irrelevant or some degree of leakage acceptable	Structure may be designed using the provisions of clause 7.3.1 of BS EN 1992-1-1. See Note A below
1	Leakage to be limited to a small amount. Some surface staining or damp patches acceptable	Width of any cracks that can be expected to pass through the full thickness of the section (i.e. cracking due predominantly to tension or restraint) should be limited to $w_{k,1}$. See Note B. Where the full thickness of the section is not cracked (i.e. cracking due predominantly to flexure), provisions of clause 7.3.1 of BS EN 1992-1-1 may be used subject to conditions in notes C and D below
2[a]	Leakage to be minimal. Appearance not to be impaired by staining	Cracks that may be expected to pass through the section should be avoided, unless special measures are incorporated (e.g. water bars or liners). There is an inference in the code that it may be adequate to provide water bars to safeguard against leakage across through cracks and verify the conditions in Notes C and D below for cracks that do not penetrate the whole depth of the section
3[a]	No leakage permitted	Special measures will be required (e.g. liners or prestress)

Key
a These are unusual requirement for basements
Notes
See BS EN 1992-3 Table 7.105 and Notes A-D below.

Note A

The UK NA to BS EN 1992-1-1 gives limit of w_{max} = 0.3 mm for reinforced concrete in all X0, XC, XD, and XS exposure Classes. For X0, XC1 exposure Classes, crack width has no influence on durability and this limit is set to produce acceptable appearance. In the absence of specific requirements for appearance the 0.3 mm limit may be relaxed. There are further requirements for prestressed members.

Note B

The values for $w_{k,1}$ in the UK NA to BS EN 1992-3 are shown in Table 9.3. The crack widths are related to h_d/h values where h_d is the hydraulic head and h is the overall thickness of the wall.

Table 9.3
Limiting crack widths for Tightness Class 1

h_d/h	≤ 5	10	15	20	25	30	≥ 35
$w_{k,1}$ (mm)	0.200	0.175	0.150	0.125	0.100	0.075	0.050

Note C

Where the whole section is not cracked, the depth of the compression zone should be checked to be at least x_{min}: in the UK, x_{min} is the lesser of 50 mm or 0.2*h*.

For sections subject only to bending, the depth of the compression zone can be calculated by geometrical parameters (i.e. section dimensions and reinforcement area: see calculation procedure in Appendix B).

For the case of combined axial tension (or compression) and bending, this axial load should also be taken into account, and should be calculated under quasi-permanent combination of loads. ψ factors for liquid retaining structures should be taken as 1.0 and operational or full depth of water should be used.

In basements, axial loads are often considered to be low and compressive and so are ignored.

Note D

The crack width limit $w_{k,1}$ in Note C above is only acceptable if the overall *range* of strain at a section under service conditions is less than 150×10^{-6}. If this is not satisfied self-healing is unlikely to occur at the crack and this may lead to leakage. (See calculation procedure in Appendix B).

9.6.2 Crack width requirements

Having established the *type* of basement construction and performance required, Table 9.4 allows the appropriate crack width requirements to be chosen taking account of the various needs.

It can be seen that Tightness Class 0 in Table 9.2 will be appropriate for *types* A and C construction (see Section 3.2).

For *type* B construction where the water table is permanently high, Tightness Class 1 will be suitable. It will be noted that the acceptable through crack widths range from 0.2 mm to 0.05 mm and are related to the ratio of the hydraulic head to the overall thickness of the wall. If the full section is not cracked, i.e. the section is subject, predominately, to flexure, then there are limits on minimum neutral axis depth and stress ranges (flexural crack widths are governed by BS EN 1992-1-1). As intimated by the note to clause 7.3.1(110) of BS EN 1992-3, when the provisions of Tightness Class 1 are met, cracks may be expected to self-heal in 'a relatively short time'. Thus a Tightness Class 1 structure may be expected to comply with the requirement of BS 8102 for 'no leakage' – in the long term. Note however that cracks may lead to staining.

With respect to *type* B construction associated with variable water tables, judgement is required. Code of Practice BS 8007[42] for the design of concrete structures for retaining

aqueous liquids prescribed a through crack width limit of 0.2 mm without any distinction related to the hydraulic head. On the basis of past experience in the UK, it is suggested that for sites where the water table fluctuates, it may be appropriate to use 0.2 mm as the limiting crack width.

When verifying for accidental water table only, it may be appropriate to use a limit of 0.3 mm for through crack width.

The adoption of Tightness Class 2 or 3 would be an unusual requirement for basement structures.

The above discussion relating in-situ concrete construction to BS 8102 and BS EN 1992 is summarized in Table 9.4. The approach to crack control and the performance implications of the chosen method should be agreed with the client.

Table 9.4
Summary of crack width requirements for different types of in-situ concrete basement construction.

Construction type[a] and water table	Expected performance of structure	Crack width requirement	Tightness Class	w_k mm	
				Flexural,[9] $w_{k,max}$	Restraint/ axial $w_{k,1}$[10]
A	Structure itself is not considered watertight	Design to Tightness Class 0 of BS EN 1992-3. See Table 9.2. Generally 0.3 mm for RC structure	0	0.30	0.30[e]
B – high **permanently high water table**	Structure is almost watertight	Design to Tightness Class 1 of BS EN 1992-3. See Table 9.2. Generally 0.3 mm for flexural cracks but 0.2 mm to 0.05 mm for cracks that pass through the section	1	0.30[b]	0.05 to 0.20
B – variable **fluctuating water table**	Structure is almost watertight	Design to Tightness Class 1 of BS EN 1992-3. See Table 9.2. Generally 0.3 mm for flexural cracks but 0.2 mm for cracks that pass through the section	1[c]	0.30[b]	0.20
B – low[d] **water table permanently below underside of basement slab but structure designed for accidental water table at ultimate limit state (strength and equilibrium)**	Structure is watertight under normal conditions. Under exceptional conditions some risk of water penetration	Design to Tightness Class 0 of BS EN 1992-3. See Table 9.2. Generally 0.3 mm for RC structures	0[c]	0.30	0.30
C	Structure itself is not considered watertight	Design to Tightness Class 0 of BS EN 1992-3. See Table 9.2. Generally 0.3 mm for RC structure.	0	0.30	0.30[e]

Key
a Construction types are defined in BS 8102 and Section 3.2 of this guide.
b Where the section is not fully cracked (i.e. the section is subject predominantly to flexure) the neutral axis depth at SLS should be at least x_{min} (where x_{min} > max {50 mm or 0.2 × section thickness}) and variations in strain should be less than 150×10^{-6}.
c See Section 9.6.2.
d 'Low' applies to free-draining strata only. This might be achieved using external drainage which is and will continue to be maintained.
e It should be noted that masonry, plain concrete and precast construction which may be considered as options for Type A and Type C forms of construction have no specific restriction to limit shrinkage cracks.

9.7 Crack width calculations

Crack control may be demonstrated by direct calculation of crack widths and compliance with the stated limits or by limiting bar size and/or bar spacing. Formulae used are shown below. They are generally based on BS EN 1992-1-1[9], BS EN 1992-3[10], their National Annexes[9a,10a] and CIRIA report C660[18].

9.7.1 Crack width and crack spacing

In all cases the crack width is calculated as the product of the crack inducing strain and the crack spacing (i.e. the movement over a length equal to the crack spacing). The relevant formulae from BS EN 1992-1-1 (Exp. 7.8, 7.11) are given below.

Crack width

$w_k = s_{r,max}\ \varepsilon_{cr}$

BS EN 1992-1-1 Exp. (7.8)

where

$s_{r,max}$ = Maximum crack spacing

= $3.4c + 0.425\ (k_1 k_2 \phi / \rho_{p,eff})$

BS EN 1992-1-1 Exp. (7.11)

where

c = nominal cover, c_{nom} in mm in accordance with BS EN 1992-1 Clause 4.4.1 and the UK NA, BS 8500[13, 14]. Often nominal covers of 50 mm to an external formed face and 30 mm to an internal face will be specified. However, c need not be taken greater than that required for durability plus the allowance made in design for deviation, Δc_{dev}[9a].

k_1 = 0.8 for high-bond bars (Note that for early age cracking calculations CIRIA C660 suggests a value of 0.8/0.7 (i.e.) 1.14 to account for poor bond conditions. BS EN 1992-1-1 Figure 8.2 defines poor conditions as affecting horizontal reinforcement in concrete either greater than 250 mm from the soffit of slabs or in the top 300 mm of members greater than 600 mm deep.)

k_2 = 1.0 for tension (e.g. from restraint)

= 0.5 for bending

= $(\varepsilon_1 + \varepsilon_2)/2\varepsilon_1$ for combinations of bending and tension where ε_1 is the greater tensile strain at one surface of the section under consideration and ε_2 is the lesser *tensile* strain (i.e. = 0 if strain at second surface is compressive).

ϕ = diameter of the bar in mm.

$\rho_{p,eff}$ = $A_s/A_{c,eff}$

This is calculated for each face. $A_{c,eff}$ for each face of a wall is based on a thickness which is the minimum of $\{0.5h;\ 2.5(c + 0.5\phi);\ (h - x)/3\}$

where

h = thickness of section

x = depth to neutral axis.

ε_{cr} = Crack-inducing strain in concrete. Defined in CIRIA C660 as the proportion of restrained strain that is relieved when a crack occurs. In BS EN 1992 terminology it is mean strain in reinforcement minus mean

strain in the concrete between cracks, i.e.
= $(\varepsilon_{sm} - \varepsilon_{cm})$.

Crack-inducing strain is derived from Sections 9.7.2 to 9.7.5 according to whether the element is subject to:

- edge restraint with
  - early thermal effects (see Section 9.7.2)
  - long term effects (see Section 9.7.3)

or

- end restraint (see Section 9.7.4)

or

- flexure and/or combinations of flexure and tension from load (see Section 9.7.5)

Note: It is assumed that the reinforcement will be at reasonably close centres. Where spacing exceeds $5(c_{nom} + \phi/2)$, BS EN 1992-1-1 Exp. (7.14) dictates that $s_{r,max} = 1.3\,(h - x)$ and PD 6687[63] requires ε_{cr} to be multiplied by $(h - x)/(d - x)$.

Where the crack width is unsatisfactory, it will be necessary to iterate on reinforcement content and/or section thickness until a satisfactory crack width is obtained.

9.7.2 ε_{cr} due to edge restraint and early thermal effects

$$\varepsilon_{cr} = K[\alpha_c T_1 + \varepsilon_{ca}]\, R_1 - 0.5\,\varepsilon_{ctu}$$

C660 Eqn. (3.6)

where

K = allowance for creep
= 0.65 when R is calculated using CIRIA C660
= 1.0 when R is calculated using BS EN 1992-3.

α_c = coefficient of thermal expansion (See CIRIA C660 for values). See Table A6 for typical values.

T_1 = difference between the peak temperature of concrete during hydration and ambient temperature °C (See CIRIA C660). Typical values for C30/37 concretes are noted in Table A7.

ε_{ca} = autogenous shrinkage strain – value for early age (3 days: see Table A9).

R_1 = restraint factor from Figure L1 of BS EN 1992-3 for the short-term situation (See Figure A2). For base-wall situations, restraint may alternatively be calculated in accordance with CIRIA C660 (See Section A5.6). Figure L1 may be used with CIRIA C660 methods providing an adjustment for creep is made (See Note 1 to Figure A2).

ε_{ctu} = tensile strain capacity of the concrete. This is a function of concrete strength and type of aggregate used. Average value(s) of 76×10^{-6} for 3 days (and 108×10^{-6} for ≥ 28 days) may be used for initial calculations until fuller mix details are known. See CIRIA C660. See Table A14 for typical values.

9.7.3 ε_{cr} due to edge restraint and long term effects

$$\varepsilon_{cr} = K[(\alpha_c T_1 + \varepsilon_{ca}) R_1 + (\alpha_c T_2 R_2) + \varepsilon_{cd} R_3] - 0.5\, \varepsilon_{ctu}$$

C660 Eqn. (3.6)

where, in addition to the definition of terms as in Section 9.7.2 above,

ε_{ca} = autogenous shrinkage strain – the long-term value for ε_{ca} should be used (see BS EN 1992-1-1, see Table A9).

T_2 = long-term or 'seasonal' drop in temperature after concreting, °C. T_2 depends on the ambient temperature during concreting. The recommended values from CIRIA C660 for T_2 are 20°C for concrete cast in the summer and 10°C for concrete cast in winter. These figures are based on HA BD 28/87[60] based on monthly air temperatures for exposed bridges. Basements are likely to follow soil temperatures So T_2 = 12°C may be considered appropriate at depth.

R_1, R_2, R_3 = restraint factors from Figure L1 of BS EN 1992-3 for short- and long-term thermal and long-term drying situations. For base-wall restraint they may be calculated in accordance with CIRIA C660. See Section A5.6. Figure L1 may be used with CIRIA C660 methods providing an adjustment for creep is made (See Note 1 to Figure A2).

ε_{cd} = drying shrinkage strain, dependent on ambient RH, cement content and member size. (see BS EN 1992-1-1 Exp. (3.9) or CIRIA C660 or Table A10). CIRIA C660 alludes to 45% RH for internal conditions and 85% for external conditions.

ε_{ctu} = tensile strain capacity of the concrete.
The 28 day value for the strain capacity should be used. For the long term (30 year or 60 year) checks, it is considered reasonable to use the 28 day value because for a more accurate answer, long term effects should be considered, but this is complicated[18]. See Table A14 for typical 28-day values.

9.7.4 ε_{cr} due to end restraint

$$\varepsilon_{cr} = 0.5\alpha_e\, k_c k f_{ct,eff}\, [1 + (1/\alpha_e\, \rho)]\, /E_s$$

BS EN 1992-3 Exp. (M.1)

where

k_c = a coefficient to account for stress distribution.
= 1.0 for pure tension.
When cracking first occurs the cause is usually early thermal effects and the whole section is likely to be in tension.

k = a coefficient to account for self-equilibrating stresses
1.0 for thickness h < 300 mm and 0.65 for h > 800 mm
(interpolation allowed for thicknesses between 300 mm and 800 mm).

$f_{ct,eff}$ = f_{ctm}
mean tensile strength when cracking may be first expected to occur:
- for early thermal effects 3 days
- for long-term effects, 28 days (which considered to be a reasonable approximation).

See Table A5 for typical values. With regard to factors that might be applicable, see definitions of $f_{ct,eff}$ in Section 9.5

α_e = modular ratio, E_s/E_c
Generally a value to 7 may be used as an average value.
Typical values are 6 at 3 days, 7 at 28 days and 12 long-term.
When cracking does occur, no creep has taken place so a modular ratio of 7 is recommended for use in crack width calculations.

ρ = ratio of total area of reinforcement to the gross section in tension.
Note that this is different from $\rho_{p,eff}$.

E_s = modulus of elasticity of reinforcing steel, which may be assumed to be 200 000 MPa.

The formula for ε_{cr} for this condition (taken from BS EN 1992-3) may be treated as an upper bound value. Verifications should be carried out for early age as well as long term using the appropriate values. The long term condition is usually considered to be critical.

In the case of end restraint, crack-inducing strain (and so crack width) is related to concrete strength and the percentage of reinforcement. The restrained strain does not influence the crack width but merely determines whether cracking will occur. Nonetheless, it is more convenient to consider the effects of end restraint in terms of strain rather than stress. See CIRIA C660 and other references for a fuller explanation.

End restraint could occur in:
- ground slab cast on piles;
- concrete panels cast as infill between pre-existing panels (alternate bay construction);
- top sections of walls in box like structures with low length/height ratio such that the base restraint will not be effective;
- slabs that are monolithic with significantly large pile caps over groups of piles; and
- suspended slabs cast between rigid walls or columns.

Where the restraint (such as that from pile caps) is local, the corresponding reinforcement may be restricted to the width of the restraint plus say 12 × slab or wall thickness.
CIRIA 660 suggests that this also applies to large areas of ground slabs with significant build up of friction. This is likely to be a cautious interpretation.

In these cases the cracks will be larger than those caused by edge restraint and end restraint is more critical for reinforcement requirements.

See footnote to 9.5

9.7.5 ε_{cr} due to flexure (and applied tension)

The procedure in BS EN 1992-1-1 should be followed. The relevant formulae are reproduced below and the code should be referred to for definition of terms.

$$\varepsilon_{cr} = (\varepsilon_{sm} - \varepsilon_{cm})$$
$$= [\sigma_s - k_t\,(f_{ct,eff}\,/\rho_{p,eff})\,(1 + \alpha_e\,\rho_{p,eff}]\,/E_s \geq 0.6\,(\sigma_s)/E_s$$

BS EN 1992-1-1 Exp. (7.9)

where

ε_{sm} = mean strain in reinforcement.

ε_{cm} = mean strain in the concrete between cracks.

σ_s = stress in the reinforcement based on cracked section properties under quasi permanent load combination (See Appendix B).

α_e = modular ratio, E_s/E_{cm}
Generally a value to 7 may be used. See above.

k_t = 0.6 for short term loading and
0.4 for long term loading.

$f_{ct,eff}$ = f_{ctm} as before at 3 days and/or 28 days.
See Table A5 for typical values. With regard to factors that might be applicable, see definitions of $f_{ct,eff}$ in Section 9.5.

$\rho_{p,eff}$ = $A_s/A_{c,eff}$
This is calculated for each face .

where

A_s = area of reinforcement provided, mm^2

$A_{c,eff}$ = Area of concrete in tension whose depth is:
minimum of $\{0.5h$ or $2.5(c + 0.5\phi)$ or $(h - x)/3\}$,
for each face of a wall [NB $2.5(c + 0.5\phi) = 2.5(h - d)$]

where

h = thickness of wall

c = nominal cover

ϕ = bar diameter

x = depth to neutral axis

d = effective depth.

0.6 = factor that limits the effect of tension stiffening.

E_s = elastic modulus for reinforcement = 200,000 MPa

9.7.6 Crack-inducing strain due to a combination of restraint effects and loading

As discussed in Section 9.1.6, it is not generally necessary to superimpose the effects of bending and direct tension with tension from restraint. In many practical cases, particularly in basement walls, stresses and strains caused by imposed deformation (shrinkage and restraint) will be at right angles to those caused by loading and therefore they need not be combined.

However, there are cases where the effects will be additive. For instance, when there is horizontal bending in a wall due to lateral loading, the stresses and strains due to imposed (shrinkage) deformation will be in the same direction and the effects will be

additive. In basement slabs that are designed as suspended slabs but cast on ground, the stresses due to restraint effects and loading will coincide and the effects again will be additive.

Where it is necessary to combine the effects of restraint and loading, the basic formula for crack width given above may be used noting that the crack inducing strain will now be the sum of the strain caused by restraint and loading effects assuming a cracked section. This approach is largely empirical but is in accordance with DD ENV 1992-1-1[64] 4.4.2.4(6). It accommodates combinations of long and short-term shrinkage and tension stiffening and eccentricities. Adding strains may be regarded as a reasonable way of estimating crack width (but it is not a valid approach to calculate moments of resistance). The outline for a (much) more detailed approach to find neutral axis depth, stresses and strains is given in Section B3.

It will be noted that cracking relieves restraint stresses and strains in adjacent uncracked sections. Besides combining edge and end restraint, which traditionally are treated separately, recent research[61] suggests that the relief of restraint strain is related to the length of the member and spacing (and number) of cracks. The formulae for edge and end restraint strain may be thought to represent extreme cases of infinitely long members. In members of finite length, it would therefore appear plausible that the relief of restraint strain limits the summation of mechanical and restraint strains. (Note that cracks do not relieve strains in uncracked sections of the same member subject to tension or constant moment.) Until this theory has been justified and developed, prudence dictates that calculated restraint and mechanical strains should be simply added.

9.7.7 Examples

By way of illustration Figures 9.11 and 9.12 show the crack widths in different thickness of walls and base slabs with different reinforcement. Figure 9.11 is for the end-restraint condition and Figure 9.12 is for the edge restraint condition.

Table 9.5 shows the crack widths predicted in a 300 mm thick element for different types of restraint and reinforcement percentages.

It can be seen clearly that for the same reinforcement, the crack width predicted in each case is different and the end-restrained elements require significant amount of reinforcement to achieve the same crack width.

Figure 9.11
End-restrained members – crack widths due to imposed deformations f_{ck} = 30 MPa.

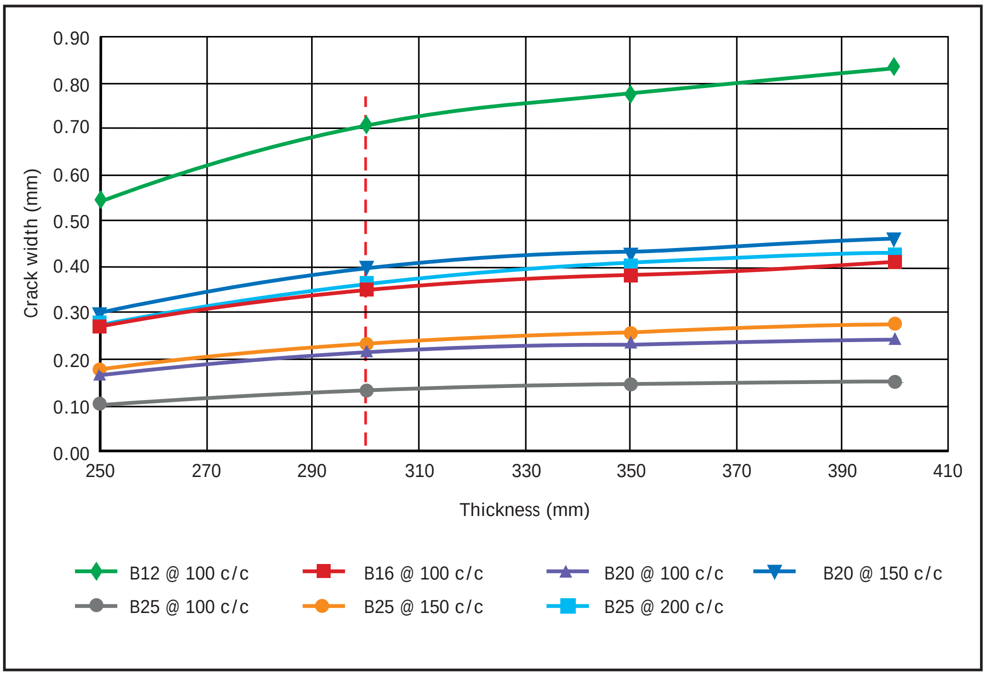


Figure 9.12
Edge-restrained members – crack widths due to imposed deformations f_{ck} = 30 MPa.

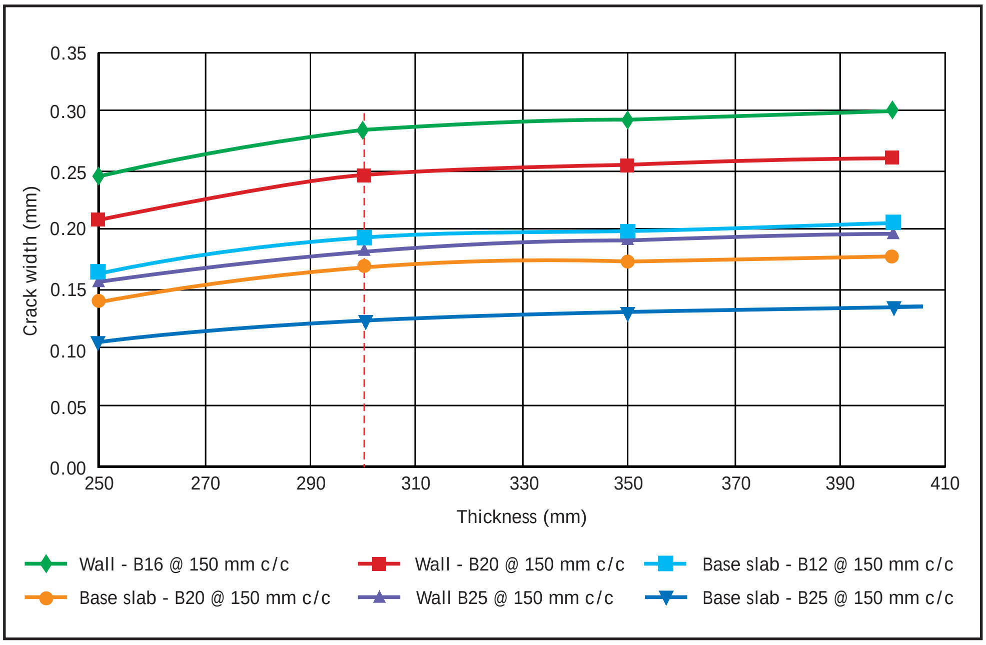


Table 9.5
Crack widths in a 300 mm thick element for different reinforcement percentages and restraint types.

Reinforcement percentage (ρ)	Edge restraint		Member with end restraint (mm)
	Wall with base restraint R = 0.73 (mm)	Base slab with restraint factor R = 0.5 (mm)	
0.89%	0.28	0.19	0.68
1.39%	0.25	0.17	0.40
2.18%	0.18	0.12	0.23

9.8 Crack control without direct calculation

BS EN 1992-3 provides charts for limiting bar sizes and bar spacing for members subject to axial tension. When cracking is dominantly caused by restraint the limiting bar sizes in chart Figure 7.103N should be used. When cracking is due to loading Figure 7.103N or 7.104N may be used.

In BS EN 1992-1-1 there are tables for the same purpose. However, it is not stated whether the tables apply to members subject to axial tension or bending. It is believed that they deal with the case of bending. The crack widths covered in the table are 0.2 mm, 0.3 mm and 0.4 mm. In the context of water excluding structures when the crack width is smaller the tables will therefore be of limited use.

In order to use either the charts or tables the stress in the reinforcement will need to be calculated. They should be based on the crack-inducing strains discussed above.

Given the above uncertainties, the calculation approach would appear to be more satisfactory.

9.9 Deflection control

In general deflections are unlikely to be critical in basement structures and the procedures in BS EN 1992-1-1 including the span/depth formulae may be used. Where finishes are applied to the structure, manufacturers should be consulted on any limitations on the strains. Base rotation may be an issue on long cantilever walls.

9.10 Minimising the risk of cracking.

BS EN 1992-3 suggests a number of strategies to minimize the risk of cracking. CIRIA C660 also provides tips for control of early thermal cracking. The following is a summary.

Materials

- Use cement replacement such as ggbs or fly ash to limit the temperature rise. Avoid pure CEM I cement.
- Use aggregates with high strain capacity. Generally they will be angular.
- Use aggregates with a low coefficient of thermal expansion.
- Use superplasticisers and water reducing agents to reduce cementitious content, but not below the level needed to achieve a good finish.
- Avoid overly strong concretes.

Construction

- Construct at low ambient temperatures to limit T_1 and T_2.
- For normal section thickness (say ≤ 500 mm) use glass reinforced plastic (GRP) or steel formwork for walls.
- Use sequential pours rather than alternate bay method, to limit end restraint.
- Where the restraint forces act parallel to the construction joint minimise the time between pours. Similarly maximize the time when the restraint forces are at right angles to the joint.

- Consider cooling of concrete.
- Avoid significant drilling and cutting of concrete after construction (difficult to seal).

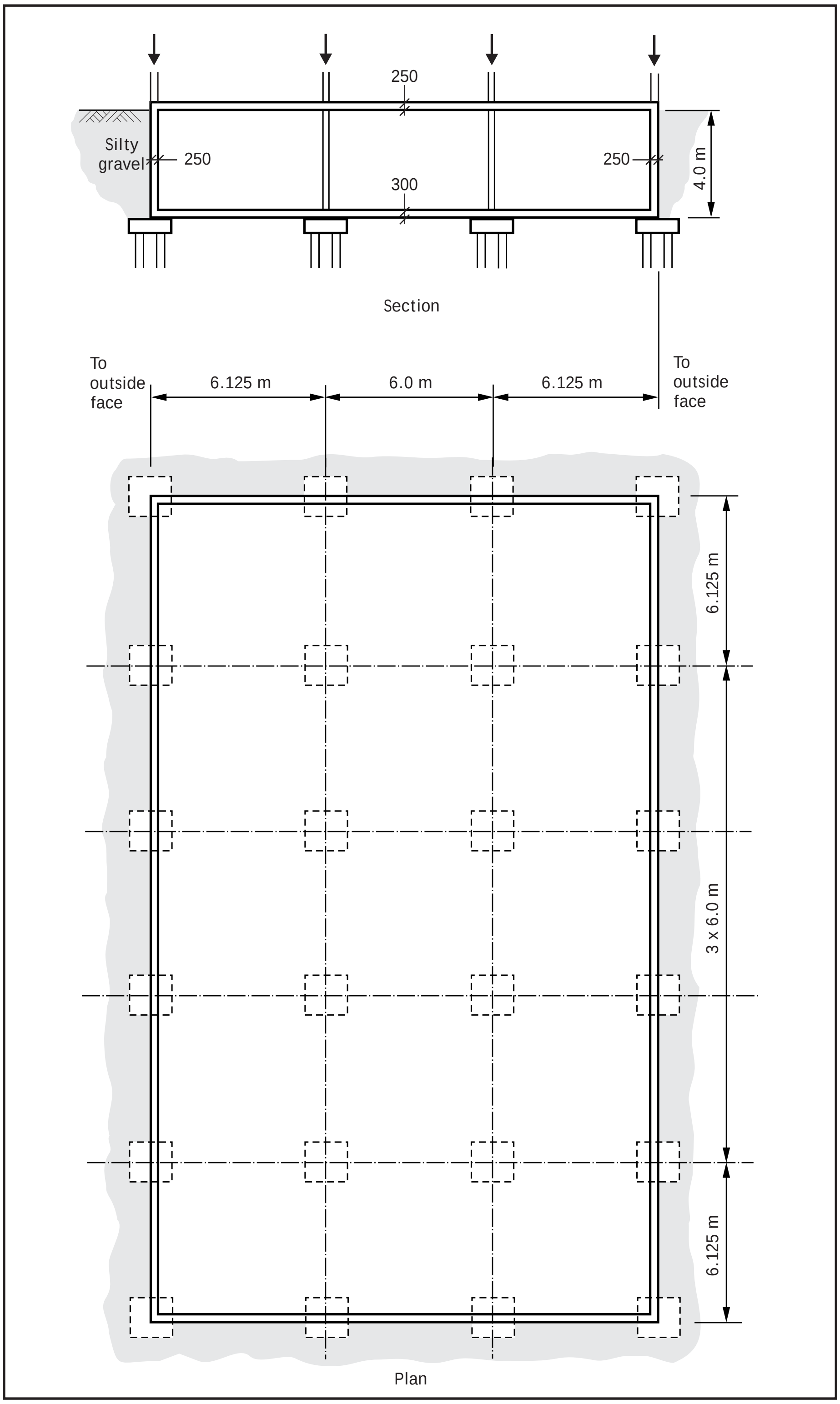
Detailing

- Use small diameter bars at close centres.
- Incorporate movement joints as a last resort.
- Incorporate prestressing if possible. Even light axial stressing will help considerably.

10. Worked example: basement slab and wall

Calculate the reinforcement required in the basement slab and walls shown in Figure 10.1 to control the crack width to 0.2 mm.

Figure 10.1
Basement general arrangement



The basement slab is 300 mm thick and will be screeded for use as offices. The basement walls and ground floor flat slab are 250 mm thick. The basement is part of a building, which is supported on piles. The basement slab is designed to span between the pile caps, which are 1.65 m × 1.65 m in plan. C30/37 concrete using a cementitious binder with 40% ggbs is proposed.

To 5 m depth, the soil is a benign well graded medium dense angular silty gravel (density 2225 kg/m^3, $N' = 21$). No water was encountered during exploratory works but ground water level was not established. Allow for a UDL = 10 kN/m^2 at ground level.

The basement is to be constructed bottom up. The basement slab is poured in two bays of 18.5 m × 15.125 m. The walls are poured in consecutive bays of approximately 12 m × 4 m.

10.1 Basement slab

mpa The Concrete Centre	Project details **Basement slab**	Calculated by CG	Job no. CCIP - 044
		Checked by PG	Sheet no.
		Client TCC	Date

10.1.1 Design for uplift at ultimate limit state

The slab needs to be designed for uplift forces.

Design height of ground water should be taken as at ground level = 4 m. The site has no natural water table. Nonetheless consider the uplift condition applying a partial factor of 1.20. The self-weight of the slab and finishes will be beneficial and a partial factor of 1.0 is appropriate.

a) Design loads

The design loads are as follows:

Uplift:

Ground water 4.0 × 10 × 1.20 = 48.0 kN/m^2

Heave (from site investigation) 0.0 kN/m^2

Self-weight of slab 0.3 × 25 × 1.0 = 7.5 kN/m^2

Weight of finishes, say, equivalent to say

85 mm of screed, 0.085 × 22 = <u>1.9 kN/m^2</u>

9.4 kN/m^2 = <u>9.4 kN/m^2</u>

= 38.6 kN/m^2

b) Equilibrium

Consider water at ground level and a central column:

Destabilising action due to water: 1.1 × 10 × 4.0 × 6.0 × 6.0 = 1584 kN/column

Stabilising action of structure:

Basement slab and finishes: 0.9 × 9.4 × 6.0 × 6.0 = 304.6

Ground floor slab: 0.9 × 0.25 × 25 × 6.0 × 6.0 = 202.5

= 507 kN/column

Presuming that the permanent action form the columns at just above Ground floor level exceeds 1584 – 507 = 1077 kN, the structure will satisfy the ULS state of equilibrium in the permanent state. Precautions may be needed during the construction phase e.g. the basement might be allowed to flood.

A similar exercise should be undertaken for the perimeter of the basement.

c) ULS bending moments

The slab is designed using bending moment coefficients.

Bending moment at support and span = $0.086\ Fl$ — Concise [56]

$F = 38.6 \times 6 \times 6 = 1389.6$ kN

$\therefore$ bending moment $M = 717$ kNm

The slab is divided into column strips of 3 m widths and middle strip of 3 m widths. The bending moment is apportioned as follows between the strips.

Table 10.1 Bending moments in column and middle strips (uplift case)

	Support (positive)	Span (negative)
Column strip (3 m wide)	0.75 × M = 537 kNm	0.55 × M = 394 kNm
Middle strip (3 m wide)	0.25 × M = 179 kNm	0.45 × M = 322 kNm

Note
BS EN 1992-1-1 Table I.1 allows some leeway on the apportionment of bending moments (and therefore reinforcement) in flat slabs.

d) Reinforcement

Cover: external (cast against blinding) 50 mm ; internal (internal) 30 mm — BS EN 1992-1-1 Cl. 4.4.1.3(4) & NA, BS 8500

Column strip – support

Assume $d = 300 - 50 - 20 - 20/2 = 220$ mm (This applies to B2, the inner (i.e. upper) layer of bottom steel; d will be greater for the outer layer).

$M/bd^2f_{ck} = 537 \times 10^6/3000 \times 220^2 \times 30 = 0.123$

$z/d = 0.883\ \therefore\ z = 0.883 \times 220$ mm $= 194$ mm. — Concise [56]

$A_s = (537 \times 10^6)/(194 \times 0.87 \times 500) = 6363$ mm^2 over the 3 m width.

For the column strip at supports, provide two-thirds in middle half of column strip i.e. provide 2828 mm^2/m:

Try H20 @ 100B (3140 mm^2/m) in middle half

H20 @ 200B (1564 mm^2/m) in outer quarters of middle strip.

Similarly, reinforcement may be calculated in other locations and is summarised below.

Table 10.2 Reinforcement for uplift case at ULS

	Support – bottom	**Span – top**
Column strip	$A_{s,req}$ = 2118 mm^2/m average H20 @ 100B (3140 mm^2/m) in middle half and H20 @ 200B (1570 mm^2/m) in outer quarters of middle strip.	$A_{s,req}$ = 1360 mm^2/m Provide H16 @ 150T (1340 mm^2)
Middle strip	$A_{s,req}$ = 656 mm^2/m Provide H12 @ 150B (753 mm^2)	$A_{s,req}$ = 1091 mm^2/m Provide H16 @ 150T (1340 mm^2)

e) Check punching shear

Retaining walls around the edges will act as beams. As a result punching shear needs to be checked only around the internal pile caps. As the slab is continuous with wall, consider that there is no elastic reaction factor to be applied to reactions to internal pile caps.

Effective punching shear = βF = 1.15 × 1389.6 = 1598 kN (relief from the area of pile cap has been ignored; but can be used).

Check at the control perimeter $2d$ using an average value of d in the two directions (i.e. (240 + 220)/2 = 230) from the face of the pile cap.

$u_O = 4 \times 1650 + 2\pi \times 2 \times 230 = 9490$ mm.

$\therefore v_{Ed} = 1598 \times 10^3/(9490 \times 230) = 0.73$ MPa

ρ_l: according to BS EN 1992-1-1 Exp. (6.47), ρ_l is the bonded tension steel in a slab width = pile cap width + $3d$ each side = 1650 + 6 × 230 = 3.03 m, i.e coincides with column strip width.

$\rho_l = 1.5 \times (3140 + 1570)/(3030 \times 230) = 7065/(3030 \times 230) = 1.01\%$

$\therefore$ from Concise table 15.6 $v_{Rd,c}$ = 0.72 MPa **$\therefore$ No good.**

Consider relief from within the loaded area (i.e. above the pile cap);

$\beta F = 1.15 \times (1389.6 - 1.65^2 \times 38.6) = 1.15 \times 1284.5 = 1477$ kN

v_{Ed} = 0.68 MPa **$\therefore$ OK and no punching shear reinforcement required.**

10.1.2 Design for download at ultimate limit state

Allow for the slab to span from pile cap to pile cap or wall.

a) Loading

Self-weight of slab 0.3 × 25 =	7.5 kN/m^2	
Weight of finishes, say, equivalent to 100 mm of screed	2.2 kN/m^2	
Imposed load (offices)		2.5 kN/m^2
	9.7 kN/m^2	2.5 kN/m^2

$n = 1.35 \times 9.7 + 1.5 \times 2.5 = 16.8$ kN/m^2

b) Bending moments

The slab is designed using bending moment coefficients.

Bending moment at support and span = $0.086\,FL$

$F = 16.8 \times 6 \times 6 = 606.4$ kN $\therefore$ bending moment $M = 312.9$ kNm

Similarly the slab is divided into column strips of 3 m widths and middle strip of 3 m widths. The bending moment is apportioned as follows between the strips.

Table 10.3 Bending moments in column and middle strips (download case)

	Support (positive)	Span (negative)
Column strip (3 m wide)	$0.75 \times M = 234.7$ kNm	$0.55 \times M = 172.1$ kNm
Middle strip (3 m wide)	$0.25 \times M = 78.2$kNm	$0.45 \times M = 140.8$ kNm

c) Design column strip – support

Assume $d = 300 - 30 - 20 - 20/2 = 240$mm (This applies to T2, inner or bottom layer of the top steel at supports).

$M/bd^2f_{ck} = 0.045 \therefore z = 0.95 \times 240$ mm $= 228$ mm. — Concise [56]

$A_s = (234.7 \times 10^6)/(228 \times 0.87 \times 500) = 2359$ mm^2 over the 3 m column strip width.

For the column strip at supports, provide two-thirds in middle half of column strip i.e. provide 1049 mm^2/m:

<u>Try H20 @ 300T (1046 mm^2/m) in middle half and</u>

<u>H16 @ 300T (670 mm^2/m) in outer quarters of middle strip</u>.

d) Design - elsewhere

Similarly reinforcement may be calculated in other locations and is summarised below.

Table 10.4 Reinforcement for vertical load case at ULS

	Support – top	Span – bottom
Column strip	$A_{s,req}$ = 789 mm^2 /m H20 @ 300T (1046 mm^2/m) in middle half and H16 @ 300T (670 mm^2/m) in outer quarters of middle strip	$A_{s,req}$ = 631 mm^2/m Provide H12 @ 150B (754 mm^2)
Middle strip	$A_{s,req}$ = 362 mm^2 /m (min) Provide H10 @ 150T (523 mm^2)	$A_{s,req}$ = 516 mm^2 /m Provide H10 @ 150B (523 mm^2)
Note Minimum reinforcement = 0.26 $(f_{ctm}/f_{yk})b_t d$ = 0.26 × (2.9/500) × 1000 × 240 = 362 mm^2		

e) Check punching shear

Effective punching shear = β F = 1.15 × 6 × 6 × 16.8 = 696 kN (Relief from the area of pile cap has been ignored; but, as before, can be used legitimately. Again assumed no elastic reaction factor.)

Check at the control perimeter 2d using an average value of d in the two directions (i.e. (260 + 240)/2 = 250) from the face of the pile cap.

u_O = 4 × 1650 + 2π × 2 x 250 = 9741 mm.
∴ v_{Ed} = 696 × 10^3/(9741 × 250) = 0.29 MPa

∴ OK by inspection

10.1.3 Design for serviceability limit state: deflection

As the uplift loading arises from an unusual design situation, deflection is not considered critical. However, using table 7.4N of BS EN 1992-1-1 the depth required is found to be 250 mm, which is slightly greater than the depth provided. This is not considered critical. If desired, additional top reinforcement in the span may be provided to bring the value of span/depth to the required value.

For the vertical load case, check end span-to-effective-depth ratio.
Allowable l/d = N × K × F1 × F2 × F3 — Concise[56]
where

N = basic effective depth to span ratio:
ρ = say 631/(220 × 1000) = 0.29%: from Table 7.4N[9] or Table NA.5[9a], N >> 26
K = structural system factor = 1.3 (end span of continuous slab)
F1 = flanged section factor = 1.0
F2 = factor for long spans associated with brittle partitions = 1.0 (span < 7.0 m)
F3 = 310/σ_s or $A_{s,prov}/A_{s,req} \leq$ 1.5 = say 1.0

Conservatively allowable l/d = 26 × 1.3 × 1.0 × 1.0 × 1.0 = 33.8

Max. span = 33.8 × 220 = 7436 mm i.e. > 6000 mm OK

10.1.4 Design for serviceability limit state: cracking

The structure is Type B and the water table is noted as being variable, so the load case described by Figure 8.1c is used. In accordance with Table 9.4 the structure will be designed as Tightness Class 1 with w_{kmax} = 0.30 mm and $w_{k,1}$ = 0.2 mm. All calculations use the formulae in Section 9.7. — Section 9.6

a) Minimum reinforcement

Minimum reinforcement to achieve controlled cracking is calculated separately for effects of restraints and cracking caused by loading. — Section 9.7

For cracking due to restraint to movement (the whole section is in tension)

$A_{s,min} = k_c\, k\, A_{ct}\, (f_{ct,eff}/f_{yk})$

$k_c = 1.0$ — Section 9.5

$k = 1.0$ for 300 mm slab

$f_{ct,eff}$ = 1.73 MPa at early age :(see Table A5 assuming cement Class N)

= 2.9MPa long term - critical - (factor for sustained loading not used)

f_{yk} = 500 MPa

$A_{s,\,min}$ = 1.0 x 1.0 x 300 x 1000 x 2.9 = **1740 mm²**

∴ require approx 870 mm^2/m T and 870 mm^2/m B.

Check minimum reinforcement requirements against design reinforcement. See Table 10.5

Table 10.5 Minimum reinforcement check

Location	Support		Span	
	Design at ULS	Minimum	Design at ULS	Minimum
Column strip	1046 & 670 mm^2/m T 3140 & 1570 mm^2 B	OK but increase H16 @ 300T (670 mm^2/m) to H20 @ 300T mm (1046 mm^2/m)	753 mm^2/m B + 1340 mm^2/m T	OK
Middle strip	362 mm^2/m T + 753 mm^2/m B	No good. Increase to H12 @ 150T (753 mm^2/m) and H16 @ 150B (1340 mm^2/m)	523 mm^2/m B + 1340 mm^2/m T	OK but increase to H12 @ 150B (753 mm^2/m) and H16 @ 150T (1340 mm^2/m)

It will be seen that the reinforcement provided for ULS in the column strip is generally OK but the reinforcement in the middle strip, particularly at support is inadequate. Therefore the reinforcement in the middle strip is increased to H16 @ 150 mm and H12 @ 150 mm

Min. reinforcement does not guarantee crack widths. Rather crack widths due to restraint to movement and loading are calculated, as illustrated in the following sections, for the reinforcement provided.

b) Check crack widths: end restraint condition

As the pile caps are substantial compared with the slab: they will restrain the slab. Consider the slab as being cracked and subject to end restraint.

Crack width, w_k $= s_{r,max}\,\varepsilon_{cr}$

Maximum crack spacing, $s_{r,max} = 3.4c + 0.425\,(k_1\,k_2\phi/\rho_{p,eff})$ — Section 9.7.4

The crack-inducing strain, $\varepsilon_{cr} = 0.5\alpha_e\,k_c k f_{ct,eff}\,[1 + (1/\alpha_e\,\rho)]\,/E_s$

Check the crack width in the (critical) top of the slab.

Consider the outer quarters of the column strip where the reinforcement provided is 1046 mm² T2 + 1570 mm² B2:

Check long-term condition

$s_{r,max}$:

$c = 30 + 20 = 50$ mm

$k_1 = 0.8$ (assume reinforcement in good bond conditions)

$k_2 = 1{,}0$

$\phi = 20$ mm

$\rho_{p,eff\,(min)} = 1046/(1000 \times \min\{300/2;\ 2.5(50 + 20/2);\ (300 - (-\infty))/3\})$

$= 1046/(1000 \times 150)$

$= 0.0070$

$s_{r,max} = 3.4 \times 50 + (0.425 \times 0.8 \times 20)/0.0070$

$= 170 + 971 = \underline{1141\text{ mm}}$

ε_{cr}:

$\alpha_e = 7$ $k_c = 1.0$ $k = 1.0$ $f_{ct,eff} = 2.9$ MPa (No factor)

$\rho = (1046 + 1570)\,/(300 \times 1000) = 0.0087$

$\varepsilon_{cr} = 0.5 \times 7 \times 1.0 \times 1.0 \times 2.9\,(1 + 1/(7 \times 0.0087))/(200 \times 10^3) = \underline{884 \times 10^{-6}}$

Assume that the tensile strain capacity $\varepsilon_{ctu} = 108 \times 10^{-6}$. (See Table A14)

$\varepsilon_{cr} >> \varepsilon_{ctu}$ ∴ Section likely to crack

$w_k = 1141 \times 884 \times 10^{-6} = 1.01$ mm > 0.2 mm required. **No good!**

By providing H20 @ T2 & B2

$\rho_{p,eff} = 3140/(1000 \times 150) = 0.0209$

$\rho = (2 \times 3140)/(300 \times 1000) = 0.0209$

$\varepsilon_{cr} = 0.5 \times 7 \times 1.0 \times 1.0 \times 2.9\,(1 + 1/(7 \times 0.0209))/(200 \times 10^3) = \underline{398 \times 10^{-6}}$

$s_{r,max} = 3.4 \times 50 + (0.425 \times 0.8 \times 20)/0.0209 = 170 + 325 = \underline{495\text{ mm}}$

And $w_k = 0.196$ mm. OK

Provide H20 @ 100 T & B (3142 mm²/m)

Checking H20@100B2 using a similar approach but cover = 50 + 20 mm results in $\rho_{p,eff}$ = 0.0209 ; ρ = 0.0209 ; ε_{cr} = 397 × 10^{-6}; $s_{r,max}$ = 563 mm and w_k = 0.223 mm	Section 9.5
For w_k < 0.200 mm would require the provision of H25@125B2, or preferably H20@90B2: $\rho_{p,eff}$ = 0.0233; ρ = 0.0221, ε_{cr} = 379 × 10^{-6}; $s_{r,max}$ = 530 mm and w_k = 0.201 mm.	
However, using k_2 (or α_{ct}) = 0.8 with $f_{ct,eff}$ and H20@100B2: ρ = 0.0209; $\rho_{p,eff}$ = 0.0209 ; ε_{cr} = 317 x 10^{-6}; $s_{r,max}$ = 563 mm and wk = 0.179 mm <u>∴OK</u>	
<u>Provide H20@100B2 (3142 mm²/m)</u>	
Check short-term condition*	
$s_{r,max}$: c = 50 mm (T2) k_1 = 0.8 (with respect to CIRIA C660, say ok) k_2 = 1.0 ϕ = 20 mm $\rho_{p,eff}$ = 3142/(1000 × min{300/2; 2.5(50 + 10)}) = 3142/(1000 × 150) = 0.0209	
$s_{r,max}$ = 495 mm	
ε_{cr}: α_e = 7 k_c = 1.0 k = 1.0 $f_{ct,eff}$ = 1.73 MPa ρ = 2 × 3142/(1000 × 300) = 0.0209	
ε_{cr} = [0.5 × 7 × 1.0 × 1.0 × 1.73 (1 + 1/(7 × 0.0209))]/(200 × 10^3) = 237 × 10^{-6}	
Assume that the tensile strain capacity ε_{ctu} = 76 × 10^{-6}. (See Table A14)	
ε_{cr} >> ε_{ctu} ∴ Section likely to crack	
w_k = 495 × 237 × 10^{-6} = 0.12 mm <u>OK</u>	
<u>Provide H20@100T2</u>	
c) Check crack widths: *edge restraint condition****	
Assume restraint is from adjacent pour(s): check crack width	
$w_k = s_{r,max}\,\varepsilon_{cr}$	Section 9.7.1
where $s_{r,max}$ = 495 mm (as above) ε_{cr}: for Early age crack-inducing strain,	
$\varepsilon_{cr} = K[\alpha_c T_1 + \varepsilon_{ca}] R - 0.5\,\varepsilon_{ctu}$	Section 9.7.2
For Long-term crack-inducing strain, restraint effects,	
$\varepsilon_{cr} = K[(\alpha_c T_1 + \varepsilon_{ca}) R_1 + (\alpha_c T_2 R_2) + \varepsilon_{cd} R_3] - 0.5\,\varepsilon_{ctu}$	Section 9.7.3
Either BS EN 1992-3 or according to it's NA [10a] as an item of NCCI, CIRIA C660 may be used to determine crack inducing strain. Out of preference CIRIA C660 is used (but for illustration the BS EN 1992-3 method is shown alongside):	NA to BS EN 1992-3

*As can be seen, where end restraint condition exists, substantial reinforcement area is required to control crack widths, making calculation of short-term end restraint and edge restraint non-critical. There is evidence to suggest that full restraint needs some distance to develop: nonetheless full restraint should be assumed.

Using CIRIA C660 [18] the following parameters can be determined (See also Appendix A)	Using BS EN 1992-3 [10] the following parameters can be determined (See also Appendix A)
$K = 0.65$	$K = 1.0$
$\alpha_c = 12 \times 10^{-6}$;	$\alpha_c = 12 \times 10^{-6}$;
$T_1 = 17.0°C$ (for equivalent wall, 300 × 1.3 = 390 mm thick assuming Class N cement (40% ggbs) in steel formwork (ref CIRIA C660; See Table A7)	$T_1 = 17.0°C$;
ε_{ca} = 15 με for early thermal effects and 50 με for long-term effects; (See Table A9 assuming 3 and 28 days respectively)	ε_{ca} = 15 με for early thermal effects and 50 με for long-term effects;
R_1 = 0.8 adjacent previous pour (see Table A12) (or 0.5 × 1.54 ≈ 0.8. See Figure A2a and notes.	R_1 = 0.5 adjacent previous pour; (See Figure A2)
$R_2 = R_3$ = (See Table A13) say 0.2 x 1/0.35	$R_2 = R_3$ = 0.20 say (See Table A13)
ε_{ctu} = 76 με for early age and 108 με for long-term effects; (C30/37 concrete, Class N cement: See Table A14)	ε_{ctu} = 76 με for early age and 108 με for long-term effects;
$T_2 = 10°C$ (See 9.7.3: winter casting assumed) and	$T_2 = 10°C$;
ε_{cd} = 336 με (See Table A10: Class N cement; assuming RH = 45% and one side exposed, ε_{cd} for 600 mm thick ≈ ε_{cd} for 500 mm thick)	ε_{cd} = 336 με
Early age $\varepsilon_{cr} = K[\alpha_c T_1 + \varepsilon_{ca}] R_1 - 0.5\ \varepsilon_{ctu}$ $= 0.65(12\ \mu\varepsilon \times 17.0 + 15\ \mu\varepsilon)\ 0.8$ $-0.5 \times 76\ \mu\varepsilon$ $= 148\ \mu\varepsilon - 38\ \mu\varepsilon$ $= 110\ \mu\varepsilon$	**Early age** $\varepsilon_{cr} = K[\alpha_c T_1 + \varepsilon_{ca}] R_1 - 0.5\ \varepsilon_{ctu}$ $= 1.0(12\ \mu\varepsilon \times 17.0 + 15\ \mu\varepsilon)0.5$ $-0.5 \times 76\ \mu\varepsilon$ $= 110\ \mu\varepsilon - 38\ \mu\varepsilon$ $= 72\ \mu\varepsilon$
Long -term ε_{cr} $\varepsilon_{cr} = K[(\alpha_c T_1 + \varepsilon_{ca}) R_1 + (\alpha_c T_2 R_2)$ $+ \varepsilon_{cd} R_3] - 0.5\ \varepsilon_{ctu}$ $= 0.65(12\ \mu\varepsilon \times 17.0 + 50\ \mu\varepsilon)0.8$ $+ 0.65\ (12\ \mu\varepsilon \times 10 \times 0.35)$ $+ 0.65\ (336\ \mu\varepsilon \times 0.35) - 0.5 \times 108\ \mu\varepsilon$ $= 132\ \mu\varepsilon + 27\ \mu\varepsilon + 76\ \mu\varepsilon - 54\ \mu\varepsilon$ $= 181\ \mu\varepsilon$ ε_{cr} = 72 με for early age and 169 με for long term effects.	**Long -term** $\varepsilon_{cr} = K[(\alpha_c T_1 + \varepsilon_{ca}) R_1 + (\alpha_c T_2 R_2)$ $+ \varepsilon_{cd} R_3] - 0.5\ \varepsilon_{ctu}$ $= 1.0(12\ \mu\varepsilon \times 17.0 + 50\ \mu\varepsilon)0.5$ $+ (12\ \mu\varepsilon \times 10 \times 0.20)$ $+ (336\ \mu\varepsilon \times 0.20) - 0.5 \times 108\ \mu\varepsilon$ $= 132\ \mu\varepsilon + 24\ \mu\varepsilon + 67\ \mu\varepsilon - 54\ \mu\varepsilon$ $= 169\ \mu\varepsilon$ ε_{cr} = 110 με for early age and 181 με for long term effects.
Long –term crack widths $w_k = 495 \times 169\ \mu\varepsilon = 0.084$ mm OK	**Long –term crack widths** $w_k = 495 \times 181\ \mu\varepsilon = 0.090$ mm OK

Note:
Calculations for the bottom of the slab would be similar.

d) Check crack widths due to combined restraint and flexure

As the piled slab is designed as a suspended slab cast on the ground and ground water level is 'variable', check combined action of restraint and flexure. As per Section 9.7.6, check by adding strains from considering restraint and flexure for the critical uplift case separately†.

ε_{cr} due to end restraint condition

As before: the crack-inducing strain due to end restraint,

$\varepsilon_{cr} = 0.5\alpha_e k_c k f_{ct,eff} [1 + (1/\alpha_e \rho)] / E_s$ — Section 9.7.4

As before : $\alpha_e = 7$, $k_c = 1.0$, $k = 1.0$, $f_{ct,eff} = 2.9$ MPa,

$\varepsilon_{cr} = 398 \times 10^{-6}$

ε_{cr} due to flexure

The crack-inducing strain due to flexure at reinforcement level:

$$\varepsilon_{cr} = (\varepsilon_{sm} - \varepsilon_{cm}) = [\sigma_s - k_t (f_{ct,eff} / \rho_{p,eff}) (1 + \alpha_e \rho_{p,eff}] / E_s$$

$$\varepsilon_{cr} \geq 0.6 (\sigma_s)/E_s$$ Section 9.7.5

where

$\sigma_s = \sigma_c \alpha_e (d - x)/x$ — (See B2.2)

$\sigma_c = M_{qp}/[A_{s2}(d - d_2)(\alpha_e - 1)\{(x - d_2)/x\} + b(x/2)(d - x/3)]$ — (See B2.2)

where

M_{qp} = BM factor × q × α Q x L^2

where

BM factor = 0.075 (see Concise[56] Table 15.4 for continuous end span)

q = 40.0 – 7.2 – 2.2
= 30.6 kN/m²

γ_Q = 1.0 [11a]*

L = 6.0 m (reduction to l_{eff} ignored)*

M_{qp} = 0.075 × 30.6 × 1.0 × 6.0² = 82.6 kN/m

A_{s2} = 3142 mm²/m

d = 240 mm to T2, as before

d_2 = 50 + 20 + 20/2 = 80 mm

α_e = 7, as before

x Using graphical method as Figure B2

A_s = 3142 mm²

ρ = 3142/(240 × 1000) = 0.0131

$\alpha_e \rho$ = 0.0916

ρ' = 3142/(220 × 1000) = 0.0143

†Not strictly correct but considered adequate for this check: See Section 9.7.6

*Quasi-permanent combinations are associated with deformations, crack widths and crack control. Characteristic combinations may be used for irreversible limit states. In this case the characteristic combination value has been used in line with the NA to BS EN 1997[11a]. It is assumed that for accompanying variable actions $\varphi_o = 1.0$.

$(\alpha_e - 1)\rho'$ = 6 x 0.0143 = 0.086

x/d = interpolating from Figure B2C = 0.344

x = 0.344 × 240 = 82.6 mm

b = 1000 mm

σ_c = $82.6 \times 10^6/[3488(240 - 80)(7 - 1)\{(82.6 - 80)/87.6\} + 1000(82.6/2)(240 - 82.6/3)]$

= $82.6 \times 10^6/[94945 + 8774873]$

= 9.31 MPa

σ_s = $\sigma_c\alpha_e(d - x)/x$

= 9.31 × 7 × (240 – 82.6)/82.6

= 124.2 MPa

k_t = 0.4 for long term

$f_{ct,eff}$ = 2.9 MPa

$A_{c,eff}$ = 1000 × min{$h/2$; $2.5(c + \phi/2)$; $(h - x)/3$}

= 1000 × min{(300/2; 2.5 × 60; 212.4/3})

= 72400 mm^2

ρ_{peff} = 3142/72467

= 0.0434

ε_{cr} = $[\sigma_s - k_t\,(f_{ct,eff}/\rho_{p,eff})\,(1 + \alpha_e\,\rho_{p,eff}]\,/E_s$

= [124.2 – 0.4 (2.9 /0.0434) (1 + 7 × 0.0434]/200 000

= [124.2 – 34.8]/(200 000)

= 447 με

Min 0.6 $(\sigma_s)/E_s$ = 0.6 × 124.2/200 000

= 370 με < 447 με

Therefore use 447 με#

Consider flexural strains at surface levels

ε_t = 447 × (300 – 82.6)/(240 – 82.6) = 617 με

ε_{crc} = 447 × (-82.6) / (300 – 82.6) = -235 με

#As the flexural strain is only to be used in combination with restraint strain, application of the minimum flexural strain may be considered conservative.

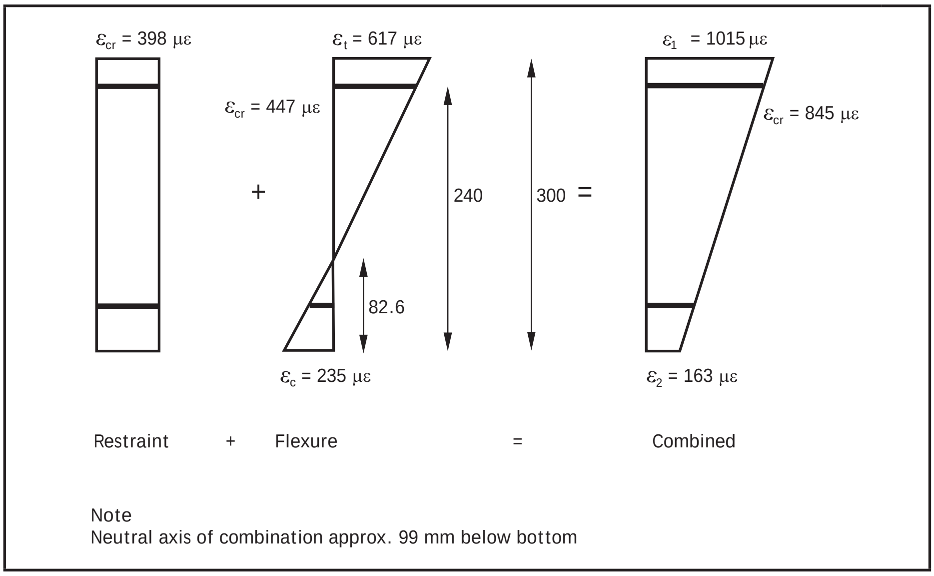


Figure 10.2 Combination of strains

Check crack spacing

$s_{r,max}$ = $3.4c + 0.425\ (k_1\ k_2\phi/\rho_{p,eff})$

where

c = 30 + 20 = 50 mm

k_1 = 0.8

k_2 = $(\varepsilon_1 + \varepsilon_2)/2\varepsilon_1$

= (1015 + 163)/(2 × 1015) = 0.580

ϕ = 20 mm

$\rho_{p,eff}$ = 3142/(1000 × min{300/2; 2.5(50 + 20/2); (300 – (–780))/3 })

= 3142/(1000 × 150) = 0.0209

$s_{r,max}$ = 3.4 × 50 + (0.425 × 0.8 × 0.624 × 20)/ 0.0209 = 170 + 189 = 359 mm

Maximum crack width

w_k = $s_{r,max}\ \varepsilon_{cr}$

= 359 × 845 με = 0.303 mm max.

Compare with limits set out for Type B construction in Table 9.3 where maximum for flexural cracks is 0.30 mm Therefore considered OK.

Average crack width = say 359 × [(1015 + 163)/2] με = 0.211 mm,

Compared with limit set out for Type B construction in Table 9.3 for assumed variable water table cracks, 0.20 mm

Considered OK

10.2 Basement walls

This section deals with the design of the basement walls. Vertical load from columns through the walls is assumed to have been dealt with elsewhere.

mpa The Concrete Centre	Project details Basement walls	Calculated by CG	Job no. CCIP - 044
		Checked by PG	Sheet no.
		Client TCC	Date

10.2.1 Design for ultimate limit state: bending in vertical plane

a) Lateral loading

Partial factors for the soil properties are required to calculate the horizontal load from soils and compaction pressures. For ULS two combinations should be considered and the partial factors are as follows.

Table 10.6 Partial Factors for ULS

	BS EN 1997-1			BS EN 1991-4
	γ_G	γ_Q	γ_φ	γ_{Qw}
Combination 1	1.35	1.5	1.00	1.20
Combination 2	1.00	1.00	1.30	1.20

Table 10.7 Design data

	Combination 1	Combination 2
Density of soil, γ	2225 kg/m^3	2225 kg/m^3
φ_{max}	30° + 4° + 4° + 2° = 40°	40°
Tan φ'_{design} = tan $\varphi_{max}/\gamma\varphi$	0.839	0.671
Tan^{-1} φ'_{design}	40°	33.8°
but φ_{crit}	38°	38°
φ'_d	38°	33.8°
$K_h = K_{ad}$ (as bottom up construction) Using charts in Annex C of BS EN 1997-1-1 or see Table A3)	0.24	0.28
K_{pd}	4.2	3.5

Section 7.3.1

Horizontal design pressures are calculated as follows:

Case 1: due to soil, imposed loads and groundwater at maximum (at ground level)

Table 10.8 Design pressures, Case 1, Combination 1.

Z (m)	q kN/m²	u kN/m²	σ'_v kN/m²	Total σ'_{ah} kN/m² $K_{ad} \times \sigma'_v$	σ'_{ah} due to variable actions (q_{ki}) kN/m²	σ'_{ah} due to water kN/m² (q_{kw}) kN/m²	σ'_{ah} due to permanent actions (g_k) kN/m²
0	10	0	10 + 0 = 10	2.4	2.4	0.0	0.0
3.85	10	38.5	10+3.85×(22.25−10.0) = 10 + 85.67 – 38.5 = 57.2	13.7+38.5 = 52.2	2.4	38.5	11.3
Partial factor on actions to be applied					1.5	1.2	1.35

Table 10.9 Design pressures, Case 1, Combination 2.

Z (m)	q kN/m²	u kN/m²	σ'_v kN/m²	Total σ'_{ah} kN/m² $K_{ad} \times \sigma'_v$	σ'_{ah} due to variable actions (q_{ki}) kN/m²	σ'_{ah} due to water (q_{kw}) kN/m²	σ'_{ah} due to permanent actions (g_k) kN/m²
0	10	0	10 + 0 = 10	2.8	2.8	0.0	0.0
3.85	10	38.5	10 + 85.67 – 38.5 = 57.2	16.0 + 38.5 = 54.5	2.8	38.5	13.2
Partial factor on actions to be applied					1.3	1.2	1.0

Table 10.10 Design pressures due to assumed 80 kg plate compactor, Case 1.

	Combination 1	Combination 2
P_d = (See Table 7.5)	14.3 × 1.35 = 19.3 kN	14.3 × 1.00 = 14.3 kN
$2P_d/\pi\gamma_{k,fill}$ =	2 × 19.3/ [3.142 × (22.25 – 10)] = 1.00	2 × 14.3/ [3.142 × (22.25 – 10)] = 0.74
Depth of point J, $z_J = (1/K_{pd})(2P_d/\pi\gamma_{k,fill})^{0.5}$ =	(1/4.2) × 1.00 = 0.24	(1/3.5) × 0.74 = 0.21
Depth of point K, $z_K = (1/K_{ad})(2P_d/\pi\gamma_{k,fill})^{0.5}$ =	(1/0.24) × 1.00 = 4.16m	(1/0.28) × 0.74 = 2.64m
Pressure at J, $\sigma'_{Jh} = K_{pd}\, z_J \gamma_{k,fill}$ =	4.16 × 0.24 × (22.25 – 10.0) = 12.4 kN/m²	3.5 × 0.21 × 12.25 = 9.0 kN/m²

The lateral loads for Case 1 are summarised in Figure 10.3.

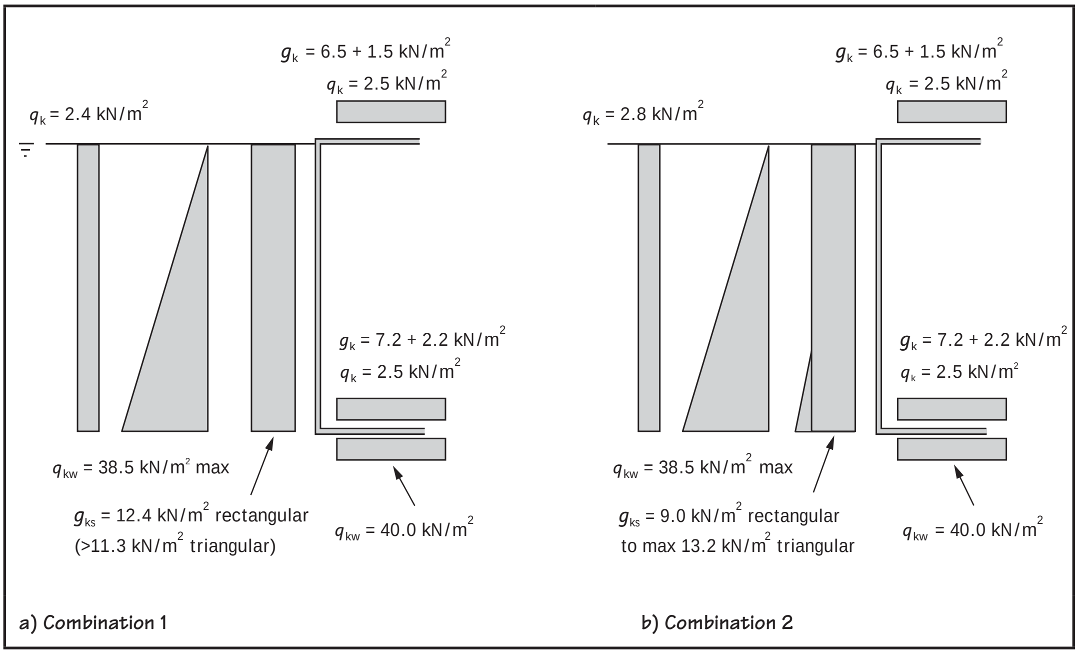


Figure 10. 3 Characteristic actions on basement wall and adjacent slabs: water at ground level.

Case 2: due to soil and imposed loads but no ground water

Table 10.11 Design pressures, Case 2, Combination 1.

Z (m)	q kN/m²	u kN/m²	σ'_v kN/m²	Total σ'_{ah} kN/m² $K_{ad} \times \sigma'_v$	σ'_{ah} due to variable actions (q_{ki}) kN/m²	σ'_{ah} due to water kN/m² (q_{kw}) kN/m²	σ'_{ah} due to permanent actions (g_k) kN/m²
0	10	0	10 + 0 = 10	2.4	2.4	0.0	0.0
3.85	10	0	10 + 85.67 = 95.7	95.7 × 0.24 = 23.0	2.4	0	20.6
Partial factor on actions to be applied					1.5	1.2	1.35

Table 10.12 Design pressures, Case 2, Combination 2.

Z (m)	q kN/m²	u kN/m²	σ'_v kN/m²	Total σ'_{ah} kN/m² $K_{ad} \times \sigma'_v$	σ'_{ah} due to variable actions (q_{ki}) kN/m²	σ'_{ah} due to water kN/m² (q_{kw}) kN/m²	σ'_{ah} due to permanent actions (g_k) kN/m²
0	10	0	10 + 0 = 10	2.8	2.2	0.0	0.0
3.85	10	0	10 + 85.67 = 95.7	95.7 × 0.28 = 26.8	2.8	0	24.0
Partial factor on actions to be applied					1.3	1.2	1.0

Table 10.13 Design pressures due to assumed 80 kg plate compaction, Case 2.

	Combination 1	Combination 2
P_d =	14.3 × 1.35 = 19.3 kN	14.3 × 1.00 = 14.3 kN
$2P_d/\pi\gamma_{k,fill}$ =	2 × 19.3/ [3.142 × 22.25] = 0.55	2 × 14.3/[3.142 × 22.25] = 0.41
Depth of point J, $z_J = (1/K_{pd})(2P_d/\pi\gamma_{k,fill})^{0.5}$ =	(1/4.2) × 0.55 = 0.13 m	(1/3.5) × 0.41 = 0.12 m
Depth of point K, $z_K = (1/K_{ad})(2P_d/\pi\gamma_{k,fill})^{0.5}$ =	(1/0.24) × 0.55 = 2.3 m	(1/0.28) × 0.41 = 1.46 m
Pressure at J, $\sigma'_{Jh} = K_{pd}\, z_J \gamma_{k,fill}$ =	4.2 × 0.13 × 22.25 = 12.1 kN/m^2	3.5 × 0.21 × 22.25 = 16.4 kN/m^2

The lateral loads for Case 2 are summarised in Figure 10.4.

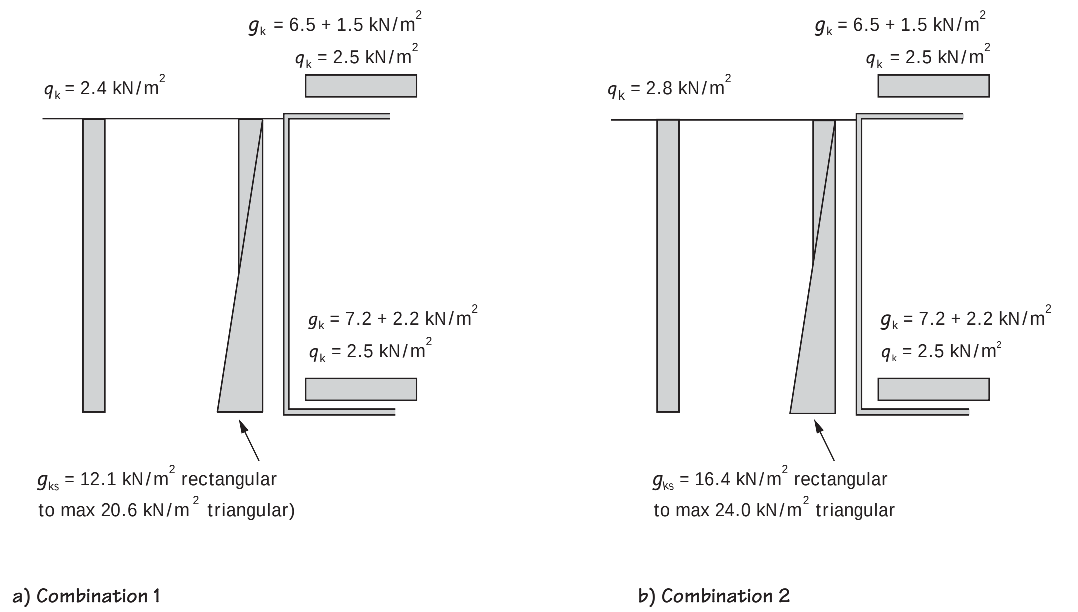
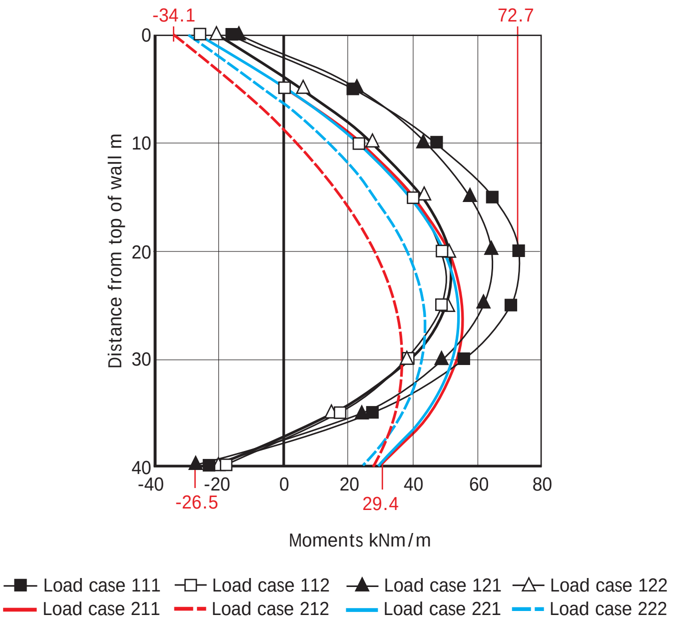


Figure 10. 4 Characteristic actions on basement wall and adjacent slabs: water below basement level

b) Vertical bending moments

Assuming backfilling only occurs following completion of the ground floor slab. The bending moment diagrams for both cases and both combinations are shown in Figure 10.5.



Notes:
Maximum moments 34.1, 72.7 and 29.4 kN/m
Maximum shear (at CL of support) = 69.3 kN/m at the top, 103.2 kN at the bottom
Load case 111 ULS Case 1: Combination 1, max. variable action from wall and uplift
Load case 112 ULS Case 1: Combination 1, max. variable action from slabs
Load case 121 ULS Case 1: Combination 2, max. variable action from wall and uplift
Load case 122 ULS Case 1: Combination 2, max. variable action from slabs
Load case 211 ULS Case 2: Combination 1,max. variable action from wall
Load case 212 ULS Case 2: Combination 1, max. variable action from slab
Load case 221 ULS Case 2: Combination 2, max. variable action from wall
Load case 222 ULS Case 2: Combination 2, max. variable action from slab

Figure 10.5 Bending moments in wall at ULS

c) Reinforcement

Design for vertical bending moment of 34.1 kNm top and bottom and 72.7 kNm at mid-height.
Cover to outermost bars 30 mm on the inside and 50 mm on the outside.
In walls horizontal bars are normally is placed on the outside. Assuming 16 mm diameter bars, effective depth for the vertical bars on the inside is

$$d = 250 - 30 - 16 - 8 = 196 \text{ mm}$$

and for vertical bars on the outside is

$$d = 250 - 50 - 16 - 8 = 176 \text{ mm}$$

At top and base

$M/bd^2f_{ck} = 34.1 \times 10^6 / (1000 \times 176^2 \times 30) = 0.037$
$z/d = 0.95$ (from Tables) Concise [56]

$\therefore A_s = (34.1 \times 10^6)/(0.95 \times 176 \times 0.87 \times 500) = 469 \text{ mm}^2$

Try H12 @ 200 mm c/c (565 mm²/m) on outside face vertically.
($\rho_l = 0.31\%$)

At about mid height

$M/bd^2f_{ck} = 72.7 \times 10^6/\ (1000 \times 196^2 \times 30) = 0.063$
$z/d = 0.94$
$\therefore A_s = (72.7 \times 10^6)/(0.94 \times 196 \times 0.87 \times 500) = 907\ \text{mm}^2\ (\rho = 0.36\%)$

Try H16 @ 200 mm c/c (1005 mm²/m) on inside faces vertically.

10.2.2 Design for ultimate limit state: bending in horizontal plane

Horizontal bending:
By inspection case 1, Combination 1 is critical for horizontal bending:
For triangularly distributed loads $n = \gamma_f g_k = 1.2 \times 38.5 = 46.2$ kN/m max.
Bending moment coefficient: (See Table A4b) consider worst case of bay width of 6 m, 12 m or 18 m i.e. k values of 1.5, 3.0 or 4.5 assuming top edge pinned. Max. coefficient for negative moments at edge, $\alpha_x = 0.037$
$M_{EdgeTri} = 0.037 \times 46.2 \times 4.0^2 = 27.4$ kNm
For rectangular loads $n = \gamma_f g_k + \gamma_f g_k = 1.35 \times 12.4 + 1.5 \times 2.2 = 46.2$ kN/m. Coefficient for long span negative moments at edge, $\beta_x = 0.032$ (See Table A4a)
$M_{EdgeRect} = 0.032 \times 20.0 \times 4.0^2 = 10.2$ kNm
$\therefore M_{Edge} = 27.4 + 10.2 = \underline{37.6\ \text{kNm}}$
Similarly, horizontal moment in mid-panel, $M_{Mid\text{-}panel}$
$= 0.012 \times 46.2 \times 4.0^2 + 0.024 \times 20.0 \times 4.0^2$
$= 8.9 + 7.7 = \underline{16.6\ \text{kNm}}$

By inspection horizontal bending is nominal

$d = 250 - 50 - 12/2 = 194$ mm

$M/bd^2f_{ck} = 37.6 \times 10^6/\ (1000 \times 194^2 \times 30) = 0.033$

$z/d = 0.95$

$\therefore A_s = (37.6 \times 10^6)/(0.95 \times 194 \times 0.87 \times 500) = 466\ \text{mm}^2$

Try H12 @ 200 mm c/c (565 mm²) horizontally on each face.

10.2.3 Shear

From analysis max shear = 103.2 kN/m at CL support at bottom.

$v_{Ed} = 103.2 \times 10^3/(1000 \times 176) = 0.586$ MPa

$\rho_l = 100 \times 565\ /\ 1000 \times 176 = 0.32\%$
$\therefore v_{Rd,c} = 0.55$ MPa $\therefore$ no good. — Concise Table 15.6
But consider at d from support where $V_{Ed} \approx 82.6$ kN; $v_{Ed} = 0.47$ MPa — 6.2.1(8)

$\therefore$ H12 @ 200 Vertically on outside face OK.

10.2.4 Design for serviceability limit state: deformation

Deflection is normally not critical in basement walls.
However, as a check, from Table 7.4N of BS EN 1992-1-1 end span/depth for $\rho \leq 0.5\%$ is > 26
∴ Effective depth required = 3850/26 = 148 mm < 196 mm used,

∴ OK.

10.2.5 Design for serviceability limit state: cracking

i) Minimum reinforcement

$A_{s,min} = k_c\, k\, A_{ct}\, (f_{ct,eff} / f_{yk})$
Using the values for the various parameters as for the basement slab the following may be obtained:
$A_{s,min}$ = 865 mm^2 for early thermal effects ($k_c\, k$ = 1.0: $f_{ct,eff}$ = 1.73 MPa, see Table A5, assuming cement Class N) 7.3.2(2)
$A_{s,min}$ = 1450 mm^2 for long-term effects ($k_c\, k$ = 1.0: $f_{ct,eff}$ = 2.9 MPa: as above)
$A_{s,min}$ = 290 mm^2 each side for bending effects ($k_c\, k$ = 0.4: $f_{ct,eff}$ = 2.9 MPa: as above)

Horizontal reinforcement = 2 × 565 is inadequate, ∴ try T12 @ 150 mm c/c on each face: i.e. total area 2 × 754 = 1508 mm^2 > 1450 mm^2)

Try H12 @ 150 bs horizontally

Vertical reinforcement = 565 + 1005mm^2/m, say OK

ii) Cracking caused by restraint to movement: edge restraint, horizontally

The procedure is similar to that used for the basement slab. Consider outer face:
crack width

$w_k = s_{r,max}\, \varepsilon_{cr}$
where

$s_{r,max}$ = 3.4c + 0.425 ($k_1\, k_2 \phi / \rho_{p,eff}$)
c = 50 mm to outer face
k_1 = 0.8
k_2 = 1,0
ϕ = 12 mm
$\rho_{p,eff}$ = 754/(1000 × min{250/2; 2.5 × (50 + 12/2); (250 - -∞ /3})
= 754/(1000 × 125) = 0.00603

$s_{r,max}$ = 3.4 × 50 + 0.425 (0.8 × 1.0 × 12 / 0.00603) = 170 + 676 **= 846 mm**

ε_{cr} = Early age crack-inducing strain,
= $K[\alpha_c T_1 + \varepsilon_{ca}]\, R - 0.5\, \varepsilon_{ctu}$ Section 9.7.2
or = Long -term crack-inducing strain, restraint effects, Section 9.7.3
ε_{cr} = $K[\alpha_c T_1 + \varepsilon_{ca})\, R_1 + (\alpha_c T_2\, R_2) + \varepsilon_{cd}\, R_3] - 0.5\, \varepsilon_{ctu}$

Either BS EN 1992-3 or according to it's NA [10a] as an item of NCCI, CIRIA C660 may be used to determine crack inducing strain. Out of preference CIRIA C660 is used (but for illustration the BS EN 1992-3 method is shown alongside):

Using CIRIA C660	Using BS EN 1992-3
Using CIRIA C660 the following parameters can be determined (See also Appendix A)	Using BS EN 1992-3 the following parameters can be determined (See also Appendix A)
$K = 0.65$	$K = 1.0$
$\alpha_c = 12 \times 10^{-6}$;	$\alpha_c = 12 \times 10^{-6}$;
$T_1 = 18.0°C$ (for a wall 250 mm thick assuming Class N cement (40% ggbs) in 18 mm ply formwork (CIRIA C660; See Table A7)	$T_1 = 18.0°C$;
$\varepsilon_{ca} = 15\ \mu\varepsilon$ for early thermal effects and $50\ \mu\varepsilon$ for long-term effects; (See Table A9 assuming 3 and ∞ days respectively)	$\varepsilon_{ca} = 15\ \mu\varepsilon$ for early thermal effects and $50\ \mu\varepsilon$ for long-term effects;
$R_j = 1/(1 + E_n A_n / E_o A_o)$ (see Appendix A5.6) where E_n (E_o) = Elastic modulus of new (old)concrete, assume $E_n/E_o = 0.80$ A_n (A_o) = Area of new (old)concrete ($\equiv h_n/h_o$) $= 1/[1 + (250 \times 0.80/300)] = 0.60$	$R_1 = 0.5$ adjacent previous pour; (See Figure A2)
$R_2 = R_3 = 0.60$ say	$R_2 = R_3 = 0.5$ max. (See Figure A2)
$\varepsilon_{ctu} = 76\ \mu\varepsilon$ for early age and $108\ \mu\varepsilon$ for long-term effects; (Class N cement: See Table A14)	$\varepsilon_{ctu} = 76\ \mu\varepsilon$ for early age and $108\ \mu\varepsilon$ for long-term effects.
$T_2 = 10°C$ (See 9.7.3: winter casting assumed) and	$T_2 = 10°C$; and
$\varepsilon_{cd} = 395\ \mu\varepsilon$ (See Table A10: Class N cement; assumed internal, 250th and as worst case two sides exposed)	$\varepsilon_{cd} = 395\ \mu\varepsilon$
Early age	**Early age**
$\varepsilon_{cr} = K[\alpha_c T_1 + \varepsilon_{ca}] R_1 - 0.5\ \varepsilon_{ctu}$ $= 0.65(12\ \mu\varepsilon \times 18 + 15\ \mu\varepsilon)\ 0.60 - 0.5 \times 76\mu\varepsilon$ $= 90\mu\varepsilon - 38\mu\varepsilon$ $= 52\ \mu\varepsilon$	$\varepsilon_{cr} = K[\alpha_c T_1 + \varepsilon_{ca}] R_1 - 0.5\ \varepsilon_{ctu}$ $= 1.0(12\ \mu\varepsilon \times 18 + 15\ \mu\varepsilon)0.5 - 0.5 \times 76\ \mu\varepsilon$ $= 116\ \mu\varepsilon - 38\ \mu\varepsilon$ $= 78\ \mu\varepsilon$
Long-term	
$\varepsilon_{cr} = K[(\alpha_c T_1 + \varepsilon_{ca}) R_1 + (\alpha_c T_2 R_2) + \varepsilon_{cd} R_3] - 0.5\ \varepsilon_{ctu}$ $= 0.65(12\ \mu\varepsilon \times 18 + 50\ \mu\varepsilon)0.60$ $+ 0.65\ (12\ \mu\varepsilon \times 10 \times 0.60)$ $+ 0.65\ (395\ \mu\varepsilon \times 0.60) - 0.5 \times 108\ \mu\varepsilon$ $= 104\ \mu\varepsilon + 47\ \mu\varepsilon + 154\ \mu\varepsilon - 54\ \mu\varepsilon$ $= 251\ \mu\varepsilon$	$\varepsilon_{cr} = K[(\alpha_c T_1 + \varepsilon_{ca}) R_1 + (\alpha_c T_2 R_2) + \varepsilon_{cd} R_3] - 0.5\ \varepsilon_{ctu}$ $= 1.0(12\ \mu\varepsilon \times 18 + 50\ \mu\varepsilon)0.5$ $+ (12\ \mu\varepsilon \times 10 \times 0.50)$ $+ (395\ \mu\varepsilon \times 0.50) - 0.5 \times 108\ \mu\varepsilon$ $= 133\ \mu\varepsilon + 60\ \mu\varepsilon + 198\ \mu\varepsilon - 54\ \mu\varepsilon$ $= 337\ \mu\varepsilon$
$\varepsilon_{cr} = 52\ \mu\varepsilon$ for early age and $251\ \mu\varepsilon$ for long term effects.	$\varepsilon_{cr} = 78\ \mu\varepsilon$ for early age and $337\ \mu\varepsilon$ for long term effects.

Crack widths:

Short -term

w_k = 846 × 52 µε = 0.04 mm ∴ OK

Long-term

w_k = 846 × 251 µε = 0.21 mm no good

H12 @ 150 inadequate

By using H12 @ 125 outside face (c = 50 mm, $s_{r\,max}$ = 734mm)

w_k = 0.189 mm ∴ OK

Using H12 @ 150 on inside face (c = 30 mm: $s_{r\,max}$ = 734mm)

w_k = 0.152 mm ∴ OK

Summary:

Horizontally use

H12 @ 125 (904 mm²/m) outside and inside faces

Crack widths:

w_k = 846 × 78 µε = 0.07 mm ∴ OK

Long-term

w_k = 846 × 337 µε = 0.28 mm no good

H12 @ 150 inadequate on outside face

By using H16 @ 125 outside face (c = 50 mm $s_{r\,max}$ = 593mm)

w_k = 0.199 mm ∴ OK

Using H12 @ 150 on inside face (c = 30 mm: $s_{r\,max}$ = 593mm)

w_k = 0.198 mm ∴ OK

Summary:

Horizontally use

H16 @ 125 (1608mm²/m) outside face and H12 @ 125 inside face

iii) Cracking caused by restraint to movement: vertically

Cracking due to restraint to movement in the vertical direction at adjacent pours may be checked in a similar manner. In this case cracking was found to be non critical. (Cover = 66 mm, R_1 = 0.5, R_2 = R_3 = 0, For H12 @ 200 w_k = 0.125 mm due to early age effects)

iv) Cracking caused by loading

Loads are calculated using the partial factors shown below.

Table 10.14 Partial factors for SCS

Partial factor	γ_G	γ_Q	γ_φ
Value at SCS	1.0	1.0	1.0

Being an irreversible limit state, cracking should be checked against the characteristic SLS load case. If ground water is treated as a variable action then the *control* of cracking should be subject to frequent or quasi-permanent SLS load case. However as control is needed when the water is at its design level the characteristic value for the load from ground water appears to be appropriate (in line with ground water being treated as a permanent action under EC7). Thus characteristic combination of loads are used in the assessment of horizontal loads due to ground water but quasi-permanent loads for vertical loads due to imposed surcharge loads.

It will be noted that the characteristic loads will be the same as for Case 1 Combination 1 at ULS (see Figure 10.3a). Worst case SLS bending moments are shown in Figure 10.6.

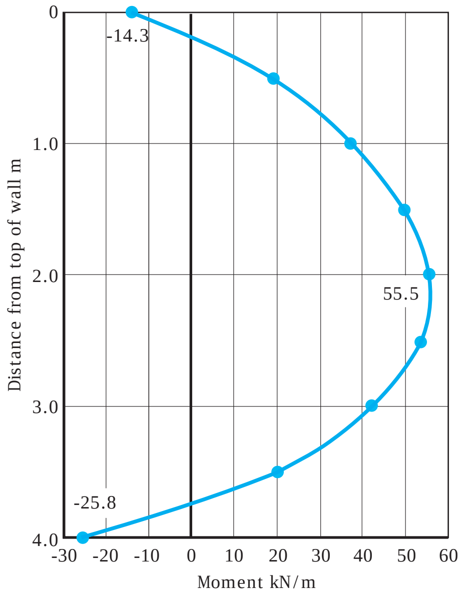


Figure 10.6 Bending moments in wall at SLS (max 55.5 kNm).

Crack width at mid height

$w_k = s_{r,max}\,\varepsilon_{cr}$

where

$\varepsilon_{cr} = (\varepsilon_{sm} - \varepsilon_{cm})$

= Crack-inducing strain

$= [\sigma_s - k_t\,(f_{ct,eff}/\rho_{p,eff})\,(1 + \alpha_e\,\rho_{p,eff}]\,/E_s \geq 0.6\sigma_s/E_s$

where

σ_s = quasi permanent tensile stress in reinforcement. Derived from formulae, via σ_c (see Appendix B)

where

σ_c = maximum quasi permanent compressive stress in concrete for a doubly reinforced section (see Appendix B2.2)

$= M_{qp}/[As_2(d - d_2)(\alpha_e - 1)\,\{(x - d_2)/x\} + b(x/2)(d - x/3)]$

where

A_s = 1005 mm^2/m (H16@200)

A_{s2} = 565 mm^2/m (H12@150)

d = 196 mm (assuming H16s each way)

d_2 = 74 mm (assuming H16s each way)

α_e = 7 (average assumed)

x = depth to neutral axis:

d_2/d = 0.377 i.e. >> 0.3 used in Figure B2c) calculate depth to neutral axis according to Section B1

= 47.6 mm#

b = breadth

= 1000 mm

#Neutral axis depth = 47.6 = 0.243 x 196. This compares to 0.235 x 196 using Figures B2c $\rho\alpha_e$ = 0.036 and $\rho(\alpha_e - 1)$ = 0.017.

σ_c = 55.5 × 10^6 / [(565(196 – 74)(7 – 1)(47.6 – 74)/47.6 + 1000(47.6/2)(196 – 47.6/3)]
= 55.5 × 10^6/[-229381 + 4287173] = 55.5 × 106/4057793 <u>= 13.7 MPa</u>

From stress diagram (see Figure B4 and Appendix B2.2)

σ_s = σ_c α_e (d –x)/x
= 13.7 × 7 × (196 – 47.6)/ 47.6 <u>= 298 MPa</u>

k_t = 0.4 for long-term loading
$f_{ct,eff}$ = 2.9 MPa (assuming α_{ct} = 1.00)
$\rho_{p,eff}$ = A_s / $A_{c,eff}$

where

A_s = 1005 mm^2 on inside face
$A_{c,eff}$ = b [min{0.5h; 2.5(c + φ/2); (h – x)/3}]
= 1000 × [min{0.5 x 250; 2.5(42+ 8); (250 – 47.6)/3}] = 1000 × 67.5 mm
$\rho_{p,eff}$ = 1005/(1000 × 67.5) = 0.015
α_e = 7 (as before)
E_s = modulus for reinforcement
= 200,000MPa

ε_{cr} = [298 – (0.4 × 2.9/0.015)(1 + 7 × 0.015)]/(200 × 103)
= (298 – 86)/(200 × 103) <u>= 1029 με</u>

$s_{r,max}$ = Maximum crack spacing
= 3.4c + 0.425 (k_1 $k_2\varphi/\rho_{p,eff}$)

where

c = 42 mm to outer face
k1 = 0.8
k2 = 0.5 (bending)
φ = 16 mm
$\rho_{p,eff}$ = 0.015

$s_{r,max}$ = 3.4 × 42 + 0.425 (0.8 × 0.5 × 16 / 0.015) = 143 + 182 <u>= 325 mm</u>

Crack width

w_k = $s_{r,max}$ e_{cr}
= 325 × 1029 με <u>= 0.33 mm - no good</u>

w_k should be ≤ 0.3 mm

<u>Try H16@200 (1005 mm²/m) on both faces</u>

Similarly to above:

x = 48.7 mm

σ_c = 13.9 MPa
σ_s = 294 MPa
ε_{cr} = 1043 με
$s_{r,max}$ = 279 mm
w_k = 0.29 mm <u>OK</u>

Neutral axis depth

As stated in Section 10.1.4 the structure should comply with Tightness Class 1 with w_{kmax} due to flexure = 0.3 mm. Note b to Table 9.3 (Note C to Table 9.2) requires that the neutral axis depth $x \geq x_{min}$ = 50 mm or 0.2 h. Say x = 48.7 is ok.

Check bar spacing:

$5(c_{nom} + \phi/2) = 5(30 + 12 + 16/2) = 250$ mm i.e. > 200 mm therefore <u>OK</u>

<u>Use Try H16@200 (1005 mm²/m) on both faces</u>

Crack width at bottom

Similarly to above using H16@200 (1005 mm²/m) on both faces:

As = 1005 mm²/m; As_2 = 1005 mm²/m; d = 176 mm, d_2 = 54 mm but x = 44.3 mm from calculation i.e. <50 mm therefore **<u>no good</u>**

<u>Try H20@200 (1570 mm²/m) on outside face at bottom of wall</u>

Similarly to above:

A_s = 1570 mm²/m
A_{s2} = 1005 mm²/m
d = 176 mm
d_2 = 54 mm
x = 52.3 mm (say OK)
σ_c = 6.3 MPa
σ_s = 104 MPa
ε_{cr} = $234_{\mu\varepsilon}$
$s_{r,max}$ = 257 mm
w_k = 0.06 mm

<u>OK Use H20@200 (1570 mm²/m) on outside face at bottom of wall</u>

By inspection of Figure 10.6, H20@200 need only extend up to starter level.

Crack width at top

By inspection crack width and neutral axis depth not critical. OK.

10.3 Summary

Reinforcement:

300 mm Basement slab: Provide H20 @ 100 T & B both ways.

250 mm Basement wall : Provide H12 @ 125 inside and outside faces horizontally (outside layers) (Design to CIRIA C660)

Provide H16 @ 200 vertically on the inside (inside layer) and on the outside (inside layer) of the basement wall.

At the bottom, provide H20@200 on the outside face up to starter level, in lieu of H16@200.

50 mm cover outside, 30 mm cover inside

Commentary

In a typical design a number of iterations would be undertaken to get the best balance of engineering judgment, economy and buildability.

In this case the reinforcement for the basement slab was found to be critical on the SLS for cracking due to long-term end restraint. The reinforcement for the walls was found to be critical on the SLS for cracking due to edge restraint. Further design effort might be put into the slab where there are estimated to be at least 53 tonnes of reinforcement compared to 10 tonnes in the walls.

With regards to cracking strain due to end restraint, the main design parameters to be looked at would be $f_{ct,eff}$, α_e and whether to apply the factor k_2 or α_{ct} to $f_{ct,eff}$. A lower strength would lower the amount of reinforcement but may have implications on buildability, durability and finish. For the reasons given in Section 5.1 a C30/37 is usually recommended. Using a modular ratio, α_e of 12 rather than the traditional 7 would increase reinforcement requirements for the assumed crack limits. It is recognized that concrete under sustained loads fails at a lower stress than under short-term loads. Recommended factors vary but taking the proposed $\alpha_{ct} = 0.8$ (see description of $f_{ct,eff}$ in Section 9.5) would reduce reinforcement requirements. In the case of the basement slab from H20@100T bw to H20@115T bw but that would be incompatible with B2 which was only justified at H20@100 (actually @108) with use of $\alpha_{ct} = 0.8$. Some rationalization and judgment is required. (Note: adopting $\alpha_{ct} = 0.8$ also affects s_{rmax}.)

New research[61] suggests that the relief of restraint strain is related to the length of the member and spacing (and number) of cracks. The formulae for edge and end restraint strain may be thought to represent extreme cases of infinitely long members. In members of finite length, some reduction in cracking strain might be adopted but this theory has yet to be fully accepted.

Additionally, the calculations to check crack width in the basement slab assumed that the groundwater was at full height and in accordance with BS EN 1997-1 used $\gamma_F = 1.0$. This is very onerous for what is considered to be an abnormal case. With reference to BS EN 1990, use of a ψ factor < 1.0 may have been more representative.

A more buildable solution for the basement slab might have been H25@150 bw T&B (3273 mm^2/m) but the balance between weight of reinforcement and fixing speed has to be judged on the circumstances.

With regards to the walls and edge restraint other design parameters might be investigated. Use of a concrete using a cementitious binder consisting of CEM I only would have made little difference to the slab reinforcement but mainly because of the difference in T_1 the outside face of the walls would have required H16 @ 125 horizontally according to CIRIA C660 or H20 @ 100 according to Eurocode 2-3. Using more than 40% ggbs may have decreased T_1 and reinforcement requirements but might have made the anticipated winter construction difficult. Better knowledge of the aggregate in the concrete may enable the use of a lower value of the coefficient of thermal expansion, α_c or modular ratio α_e. In the slab, considering ε_{cd} and drying from one side only would

have decreased requirements. However, considering summer construction would have increased T_2 to 20°C and consequently, would have increased reinforcement.

It is interesting to note that the requirement for neutral axis depth $x \geq x_{min}$ = 50 mm or 0.2h in BS EN 1992-3 was critical for the walls in vertical bending.

Out of preference the CIRIA C660 method was adopted for the design for edge restraint rather than a strict interpretation of BS EN 1992-3. In the UK this is allowable as CIRIA C660 is cited as NCCI in the UK National Annex[10a]. CIRIA C660 gives much guidance, is considered to be more up-to-date than BS EN 1992-3, relates to UK experience and appears less onerous. The main differences are the factor K that to allow for creep and the ability to calculate edge restraint according to the formula presented in Section A5.6.

There will be other construction issues that will need to be resolved – preferably in consultation with the construction team - and these may have effects on the design assumptions made and the next design iteration.

11. Specification and construction details

11.1 Specification

A project-specific specification for works should be prepared. Useful guidance may be found in *National Structural Concrete Specification for building construction*[65], BS EN 13670 – *Execution of concrete structures*[66] NBS[67] and where piling is involved *ICE specification for piling and embedded retaining walls*[68].

Good workmanship is essential to minimise defects. SPECC[69] has a specialist category for specialist basement contractors. Several organisations can provide warranties and/or guarantees but the client must balance risks against cost. Water-resisting construction can be achieved by appropriate design, careful planning, good workmanship and appropriate supervision.

Specifications will call for the contractor to provide details of proposed methods and materials, including water stops, concretes, though ties, construction joints, pour sizes and curing and for these to be agreed with the contract administrator before work commences on site. The method statement must be agreed and must be checked against the assumptions made in the design for consistency.

11.2 Joints

Joints are potentially vulnerable locations for water penetration. In basement structures, the number of joints should be kept to a minimum. There are essentially two types of joints, namely, those required for minimizing the risk of cracking (movement joints) and those required for convenience of construction (construction joints).

Movement joints

Expansion joints should be provided when reversible movements are expected and contraction joints are suitable when only contraction has to be accommodated. Basement structures will generally be in a stable environment with only small fluctuations in temperature. As such, provision of movement joints is not normally considered necessary and it is recommended that the structure is considered fully continuous and reinforced accordingly assuming that imposed deformation (e.g. shrinkage) might be fully restrained. In-service performance of such a design will be far more satisfactory compared with one that incorporates movement joints. BS EN 1992 states that movement joints should be provided when effective and economic means cannot otherwise be taken to limit cracking. **In other words movement joints are a strategy of last resort. The risk of leakage increases when they are incorporated and these joints also require maintenance for continued good performance.**

When movement joints are required, they should be protected by water bars and preferably sealed. The selection of sealants should be undertaken by a specialist and should take into account the chemical compatibility with other materials or soil with which they are likely to be in contact, expected movement in the joint and ease of repair and replacement.

Construction joints

Construction joints will, however, be required for convenience of concreting and structural continuity is assumed at these joints. The size of individual pours will be governed by the amount of concrete that can be placed in a working day and may be limited by considerations such as site constraints, ease of access for concreting, the geometry of the element, the type of finish required and so forth. *National Structural Concrete Specification*[65] recommends the pour sizes in Table 11.1 as a benchmark for discussion. Larger areas should be acceptable where the contractor provides a satisfactory method statement and provides design checks with respect to restraints on the proposed bay layout and provides any additional reinforcement required. Larger pours require larger and well managed resources. Longer construction joints lead to larger areas being subject to early age edge restraint. Sequential pours are to be preferred to infill panels: even then end restraint conditions may be difficult to avoid.

Table 11.1
Basic pour sizes

Element	Maximum area (m^2)	Maximum dimension (m)
Water-resisting walls	25	5
Water-resisting slabs	100	10
Slabs with significant restraints at both ends	100	13
Slabs with significant restraint at one end only	250	20
Slabs with minimal restraint	500	30
Walls	40	10

Note
From NSCS[65]

The designed reinforcement should pass through construction joints to provide structural continuity and to ensure that there is no relative movement between the sections. The surface of the first pour should be roughened to increase the bond strength and to provide aggregate interlock. Powerful hammers should not be used as, besides Health and Safety implications, they may dislodge the aggregate particles.

All joints in basement construction should be protected, normally by water bars, even when the water table is not present. Protection at these joints is illustrated in Figures 11.1 to 11.6.

Water stops

Section 5.9 of the NSCS[65] states that water stops should be provided at all construction joints in water-resisting construction unless specified otherwise. Water stops that are used in UK industry are described in Section 5.3 of this document. Typical details used in the UK are shown in Figures 11.1 to 11.5. Figures 11.1 and 11.2 illustrate the application of external PVC water bars. Figures 11.3 and 11.4 show typical details for the use of internal and preformed metal strips in horizontal and vertical joints in walls. Figure 11.5 illustrates potential uses for hydrophilic water bars. Figure 11.6 shows the rudiments of an injection system.

Where proprietary water bars are used, manufacturer's recommendations should be followed for the preparation of joints and installation of the water bars.

Re-injectable water stops give some assurance that construction joints can be sealed should they leak. These water stops have lead to a different way of approaching the construction of some basements. In such cases, the slab and walls are cast in small panels or with crack inducers to ensure that cracks can only occur at the construction joints or crack inducers, where centrally placed re-injectable hose has been placed. If and when cracks occur, they are simply re-injected. The amounts of crack control reinforcement required are reduced.

Figure 11.1
Typical detail of pre-formed PVC strip backstop water bar.

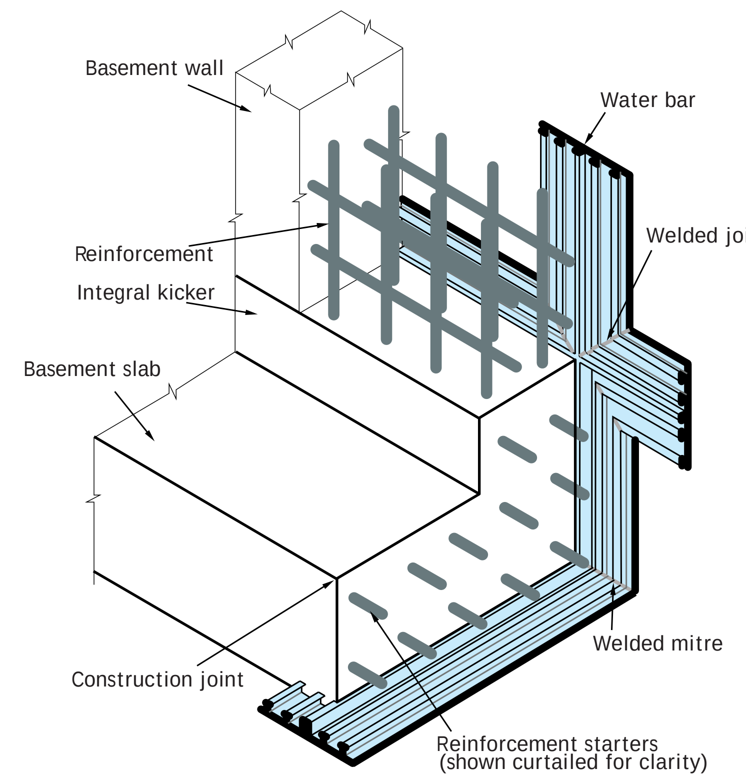
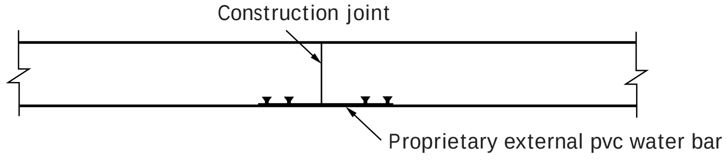


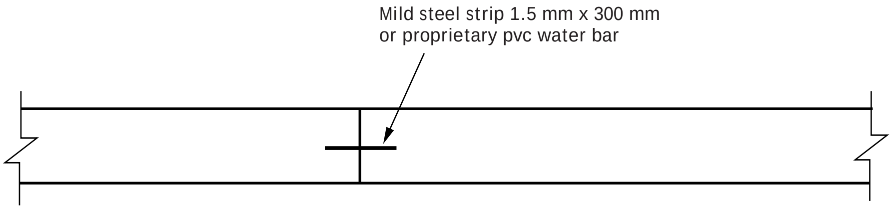
Figure 11.2
Typical construction joint in a base slab.



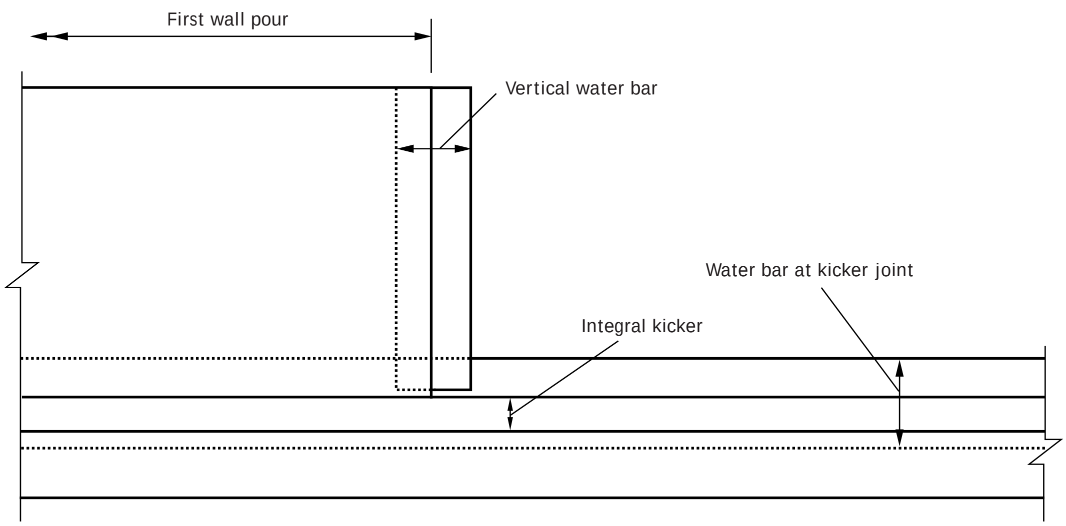
Reinforcement omitted for clarity
Expansion and contraction joints are similar except the reinforcement is not continued across the joint. At expansion joints a suitable gap is left between adjacent pours to permit expansion

Section

Figure 11.3
Typical details for using internal water bar at construction joints.

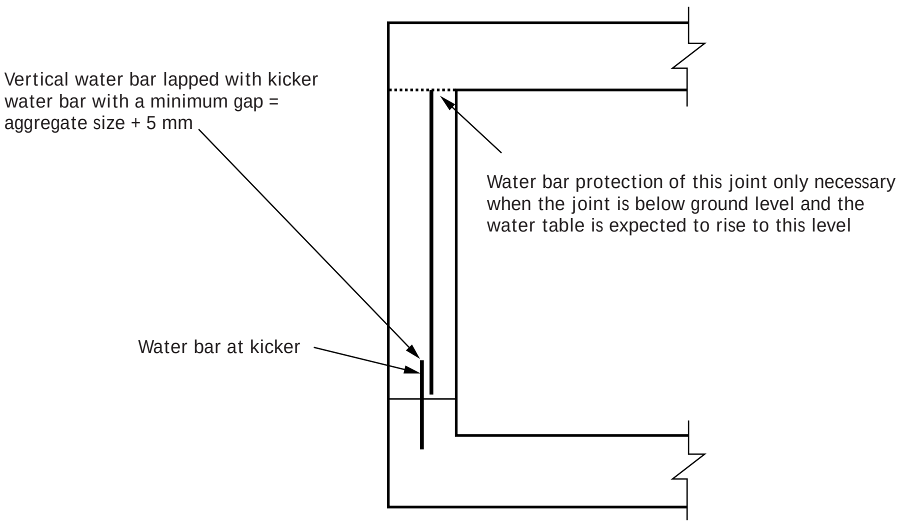


a) Plan section through wall



Reinforcement omitted for clarity
Expansion and contraction joints are similar except the reinforcement is not continued across the joint.
At expansion joints a suitable gap is left between adjacent pours to permit expansion

b) Elevation



Reinforcement omitted for clarity
Expansion and contraction joints are similar except the reinforcement is not continued across the joint.
At expansion joints a suitable gap is left between adjacent pours to permit expansion

c) Vertical section

Figure 11.4
Typical details for using metal water bar at construction joints.

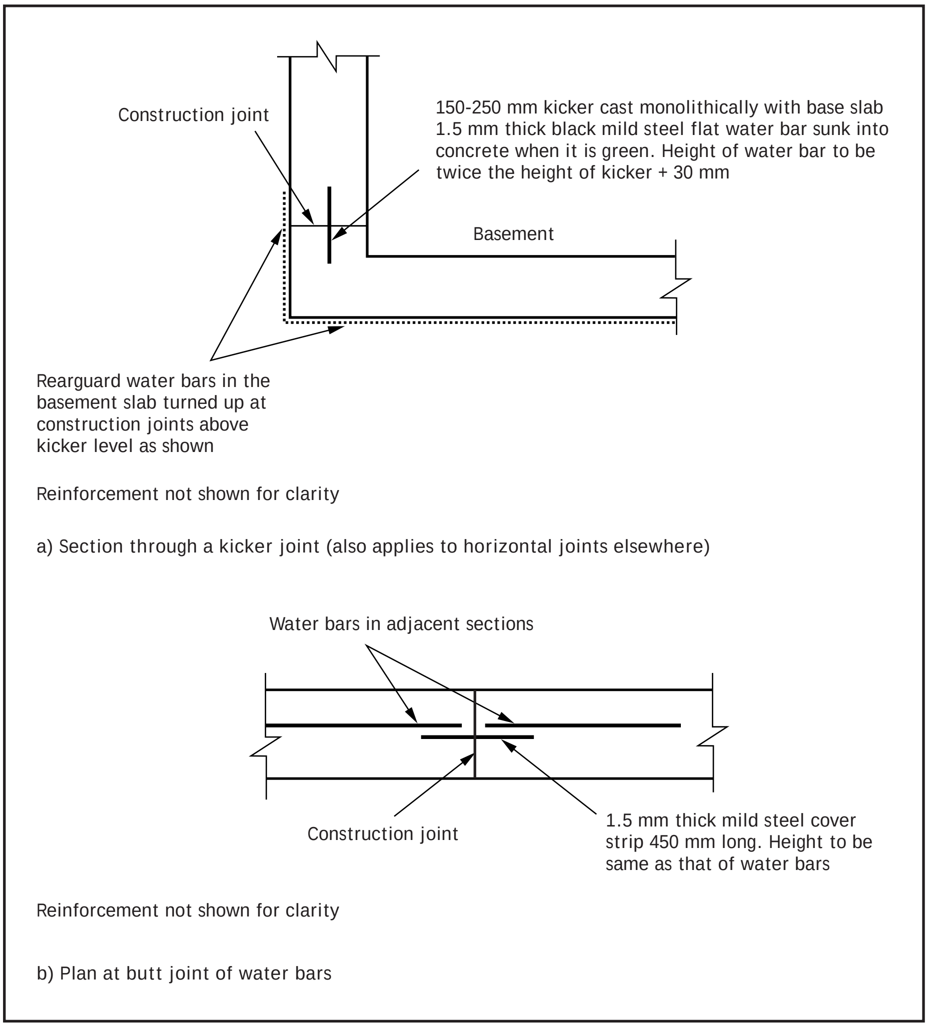


Figure 11.5
Potential locations for hydrophilic water stop.

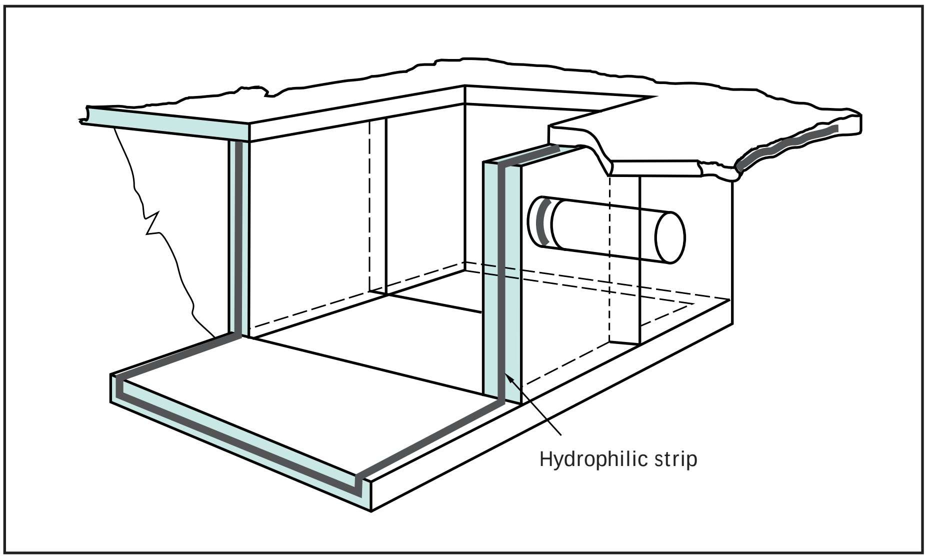


Figure 11.6
Schematic of injection hose system.
Note: Hoses should be located in the middle of section and fixed securely. The shutter connectors must be accessible to allow resin injection(s) at a later date.
Photo: Max Frank Ltd

11.3 Miscellaneous

11.3.1 Kickers

In basement construction it is recommended that a kicker with a height of between 150 mm and 250 mm is constructed to be *monolithic* with the base slab. Kickerless construction may be preferred by some contractors in the interest of speed of construction, but carries an additional risk of water penetration which is not warranted in basement construction.

Cast monolithically with the basement slab, the basement wall kicker shown in Figure 11.7 has been prepared and hydrophilic water stop fixed ready for wall reinforcement and formwork.

Figure 11.7
Integral kicker
Photo: McCoy Engineering

11.3.2 Formwork ties

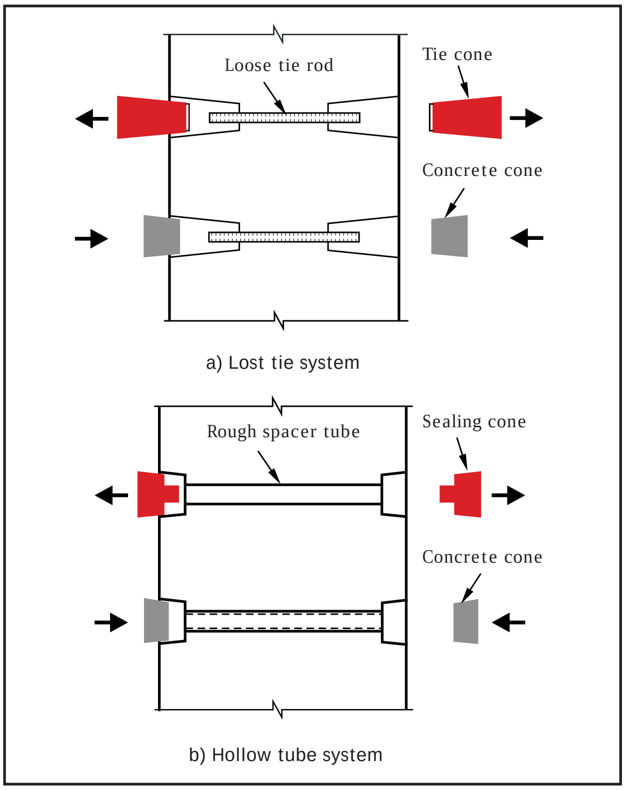
In order to minimise water ingress through in-situ reinforced concrete walls, special attention should be paid to formwork ties. There are two generic types of ties that are appropriate to watertight construction. These are shown in Figure 11.8.

In the lost tie system part of the tie, which is used to secure the two shutters, is permanently lost in the wall, and the conical voids left at the ends are then grouted up or plugged with pre-made concrete cones.

In the hollow tube system the two faces of the formwork are spaced apart using a hollow tube through which a threaded rod is passed and secured against the outer faces of the formwork. After concreting a hole is left in the wall together with conical ends. These will then be grouted up or plugged with pre-made concrete cones.

The lost tie system is better for water-resistant construction although the hollow tube system is commonly used despite the risk of water ingress.

Figure 11.8
Types of formwork ties for use in watertight construction.



11.3.3 Waterproofing membranes and coatings

These are described in Section 5.2 and should be selected and applied in accordance with the manufacturer's recommendations.

Membranes may be multi-layer or single. Especially in the case of single layer membranes, joints must be sealed not just lapped to obtain water resistance.

11.3.4 Water-resisting admixtures or additives

Water-resisting construction can be achieved by appropriate design, careful planning and good workmanship without reliance on admixtures or additives. Where specialised admixtures as described in Section 5.2 or additives are used, it is important that there is appropriate supervision.

11.3.5 Service penetrations through the structure

Service pipes penetrating the concrete in basements should be avoided wherever possible. These are common locations where leakage occurs and, if unavoidable, should be pre-planned and designed in a similar manner to penetrations in water-retaining structures. Pipes should incorporate a puddle flange (effectively incorporating a pre-welded water bar on the outer face). The perimeter should also be sealed using a hydrophilic water stop as an additional precaution.

Casting a sleeve or boxing and locating the pipes subsequently will result in more interfaces and will increase the risk of leakage (even though hydrophilic water stops are usually wrapped around the pipes) and should ideally be avoided. Post-drilling concrete and any internal membranes for fixings or service pipes will damage the integrity of the concrete and the membrane in the local area. Reinforcement may also be encountered and it may be cut or lead to relocation of the service or its fixing which will increase the area of damaged concrete. This is not a satisfactory procedure in water-excluding basements.

11.3.6 Tolerances for piling

Allowance should be made for at least normal tolerances unless agreed otherwise normal tolerances[68] used are as follows:

Setting out tolerance at ground level	± 75 mm
Inclination	1 in 75

Setting out tolerances may be reduced to 25 mm where a guide wall is used.

11.3.7 Capping beams

Capping beams to contiguous piled walls are at the hub of many design, space planning and construction issues (see Figure 3.8). The capping beam will often be required before excavation can begin. It must be designed for both temporary horizontal forces and for permanent vertical forces from structures above. Vertical forces will be spread over several piles. There is usually reinforcement on all four faces so there will be inevitable tolerance issues and clashes with pile reinforcement. It is preferable to limit bars in piles to 20 mm diameter to allow adjustments to be made and to keep anchorage lengths reasonable. There is a need to allow for starter bars for subsequent slabs, temporary connections, working space, services in adjacent ground, continuity of damp-proof

membranes and waterproofing membranes, etc. The alignment of the face of the capping beam with internal walls or facing walls should be carefully considered, particularly if a fair-faced finish is required.

The design of capping beams is discussed in more detail by Wells[70].

The capping beam shown in Figure 11.9, allows for its use as a waling to props in the temporary condition and in the final condition the side is flush with the face of the facing wall.

Figure 11.9
Capping beam.
Photo: GCL

11.3.8 Drainage

Particular care must be taken to ensure that drainage systems are properly designed and constructed to deal with the anticipated ground conditions, with access for cleaning of drains and voids and maintenance of pumps, etc. where appropriate.

11.3.9 Underpinning

Where underpinning is required reference should be made to more specialist literature such as *Design and construction of deep basements*[35].

Figure 11.10
Start of ground floor formwork.
This photograph was taken some weeks after that shown on the front cover. It shows the start of the system formwork to the ground floor of this nine storey RC frame with basement car park. As may be seen in the foreground, A-frames were used for the single-sided formwork for the basement walls.
Photo: Northfield Construction Ltd

11.4 Inspection, remedials and maintenance

Inspection

Unlike water-retaining structures, it is rarely, if ever, feasible to conduct tests for watertightness on completed basement structures. The vulnerable face for water penetration may be inaccessible in many cases. Workmanship in construction is therefore critical, and will be assisted by formal procedures and inspections.

The following is recommended:

- Detailed method statements should be prepared for each stage of construction. They should cover such items as concrete mixes, method of placing, preparation of surfaces, jointing, installation of water stops and so forth.
- All materials should be tested and certified.
- Supervision of construction is essential.

Remedial measures

In view of the nature of the work, it is usually prudent to allow for some remedial works. Even in generally well-constructed projects, some minor local making good and repairs may be necessary and these should be appropriately addressed in conjunction with the designer. Minor blowholes are best left alone. More significant blowholes and honeycombed areas should be cut back to sound concrete and filled with a grout. There are a number of proprietary repair compounds available for this purpose.

Where a leak occurs it should be carefully investigated, but tracing of the source can be laborious and time consuming. Cracks may heal by autogenous healing (see below). Where cracks prove unacceptable, remedial works will be required. Minor cracks can be injected with an appropriate sealing material.

Often water ingress will occur at joints or service penetrations. In these cases, concrete will need cutting out locally and sealing with proprietary compounds.

Remedial measures such as pressure or vacuum grouting and injection, sealing with resin or cementitious mortar and replacement of defective material are covered in BS 8102.

Autogenous healing

Autogenous healing or self sealing is defined as a natural process of closing and filling cracks in concrete (or mortar) whilst it is kept damp. In water tanks, it was at one time relied upon to produce watertight concrete.

The process involves water reacting with the free cement in the concrete to form calcium salts, which together with trapped particles and over time seal the cracks. An appreciable head is essential to force the water into the cracks for the process to occur. Cracks of up to 0.5 mm have been known to self seal. Above this crack width and/or with a head in excess of 6.0 m, the free cement present in the concrete can be leached or washed out of the concrete thereby preventing self sealing.

Autogenous healing may not be effective if there is insufficient head to force the water into the cracks and should not usually be relied upon for basement construction.

Maintenance

The design of Type C construction must provide for future access to enable cavities and drains to be cleaned and pumps to be maintained regularly so that they function efficiently.

The owners of basements should be provided with a manual containing the full details of construction, materials used, routine maintenance and periodic replacement required.

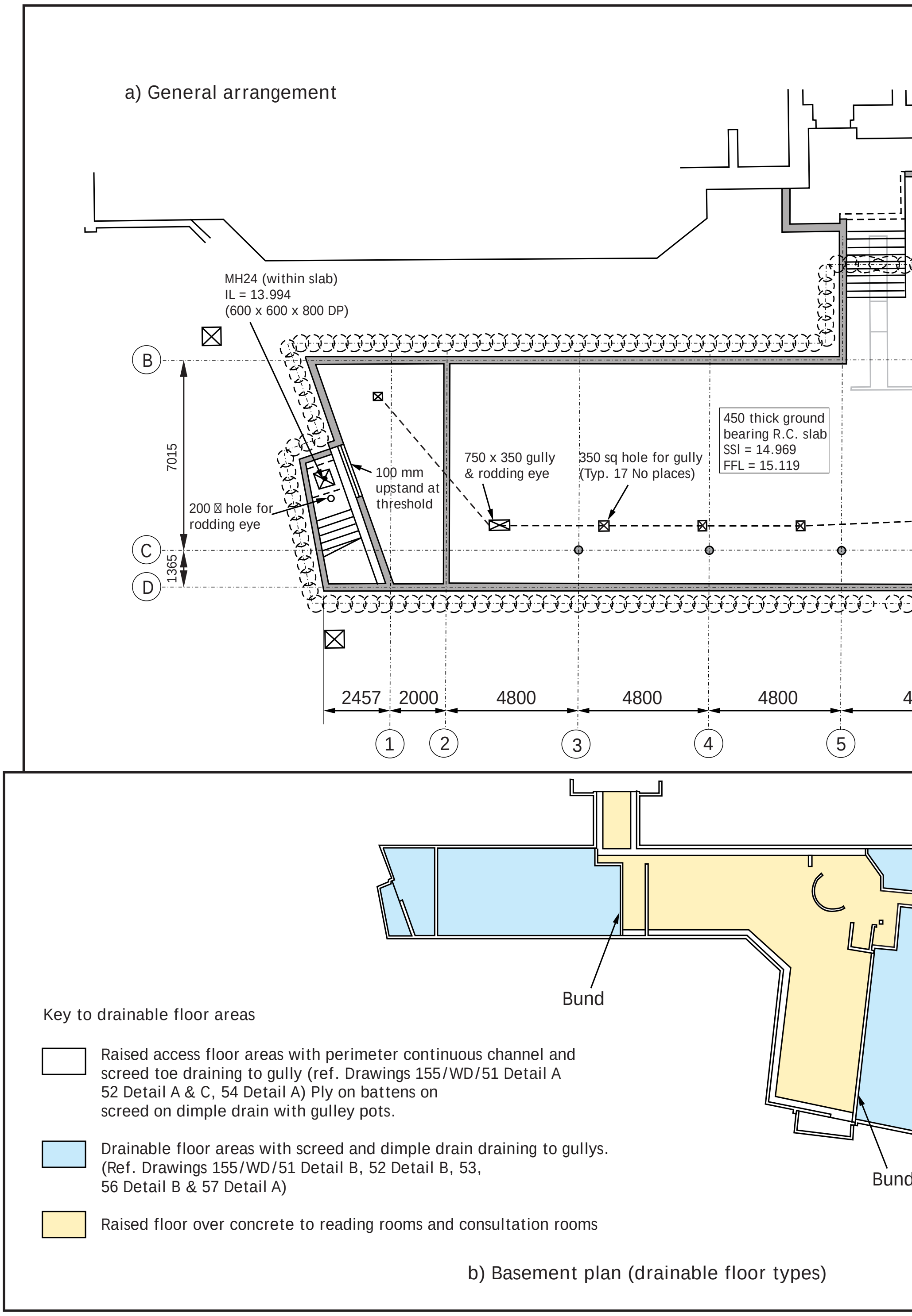
Figure 11.11
Basement construction in Nottingham.
Here it was possible to construct the basement with a cut clean out of sandstone. No earthwork support was needed although netting was used to stop loose material falling on battered sides. A-Frames were used to form perimeter RC walls.
Photo: Northfield Construction Ltd

12. Case studies

The case studies illustrated relate to construction aspects of two large basement projects which both involve propped piled walls with an internal reinforced concrete box. The projects represent different grades of intended use.

12.1 Case study 1: Basement in South-West London, 2002

Figure 12.1
General arrangement: Basement layout.



Salient features

- 4 m deep basement (depth of excavation about 5 m)
- Use – archives, exhibition and public spaces
- Soil – gravels
- Water table about 1 m below ground level
- Propped secant piling to facilitate excavations
- Concrete box designed to be inside the secant piles
- Vapour barrier membrane sandwiched between the piles (faced with polystyrene) and the concrete box
- Drained cavity inside walls and above floor

By courtesy Clark Smith Partnership

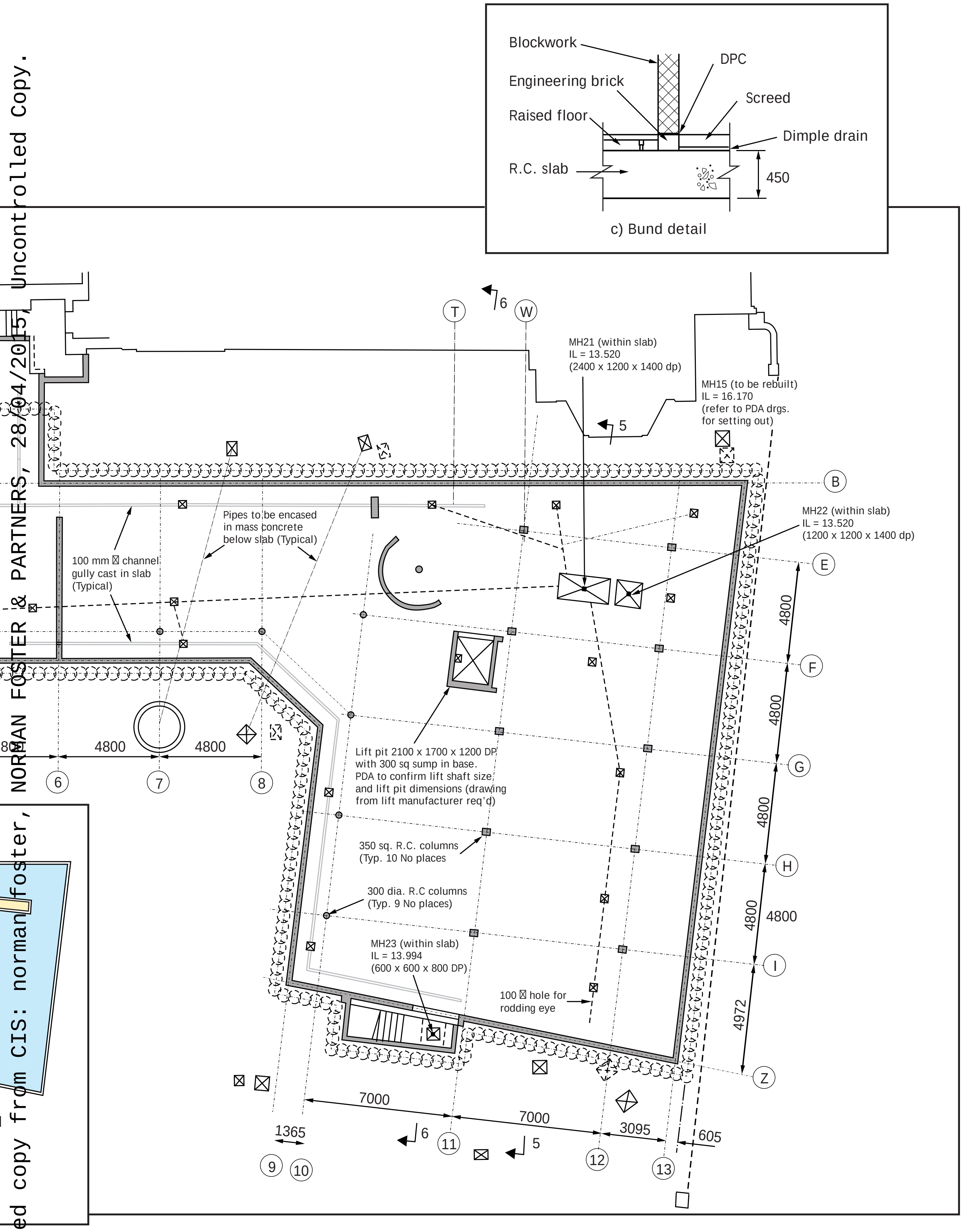
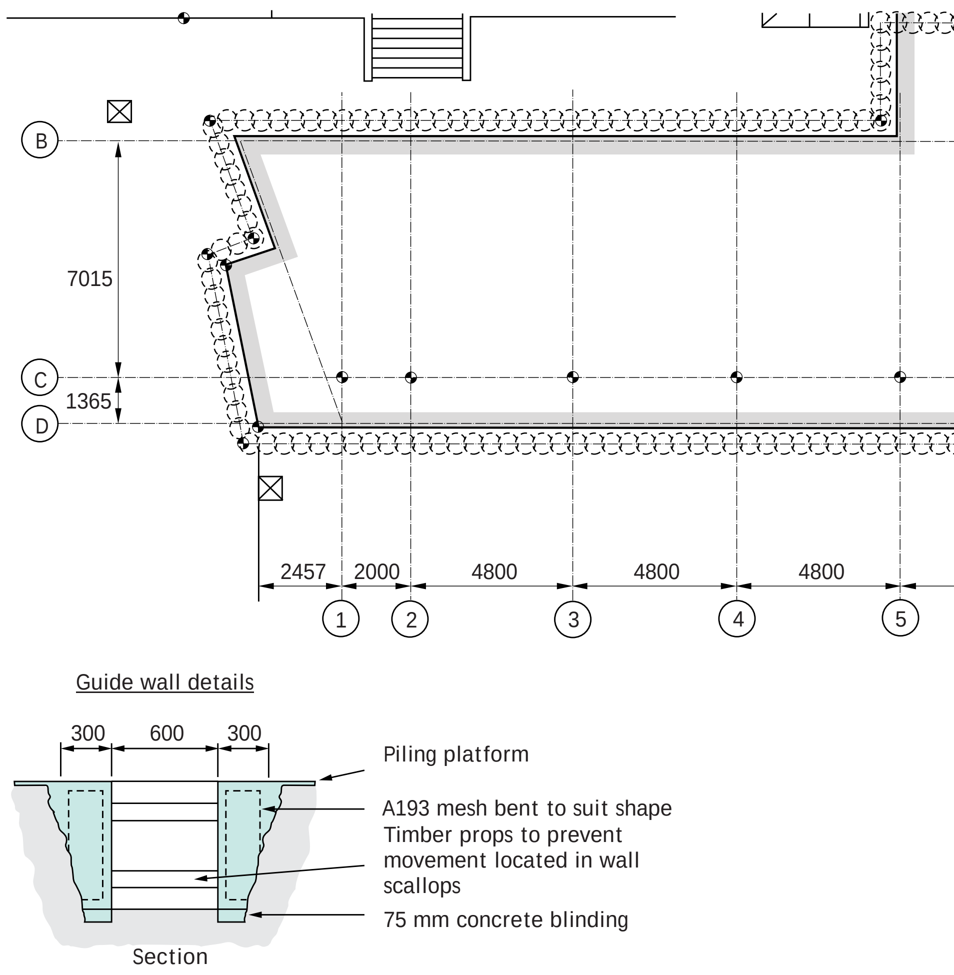

Blockwork
DPC
Engineering brick
Screed
Raised floor
Dimple drain
R.C. slab
450
c) Bund detail
MH21 (within slab)
IL = 13.520
(2400 x 1200 x 1400 dp)
MH15 (to be rebuilt)
IL = 16.170
(refer to PDA drgs.
for setting out)
MH22 (within slab)
IL = 13.520
(1200 x 1200 x 1400 dp)
Pipes to be encased
in mass concrete
below slab (Typical)
100 mm channel
gully cast in slab
(Typical)
Lift pit 2100 x 1700 x 1200 DP
with 300 sq sump in base.
PDA to confirm lift shaft size
and lift pit dimensions (drawing
from lift manufacturer req'd)
350 sq. R.C. columns
(Typ. 10 No places
300 dia. R.C columns
(Typ. 9 No places)
MH23 (within slab)
IL = 13.994
(600 x 600 x 800 DP)
100 hole for
rodding eye
4800
4800
4800
4800
4800
4800
4800
4972
7000
7000
3095
605
1365
T
W
B
E
F
G
H
I
Z
6
7
8
9
10
11
12
13
5
6

Figure 12.2
Pile layout and setting out



Piling notes:

1. Individual pile locations indicative only.
   Construction drawing to be produced by piling sub-contractor.
2. Setting out co-ordinates for information only.
3. Size of capping beam assumed to be 800 wide x 600 deep.

Notes:

1. Finished face of the guide wall to be vertical within 1:200
2. The minimum diameter of the profile should be pile diameter + 25 mm with a tolerance of + 15 mm - 0 mm
3. Concrete strength to be a minimum of 25 N/mm^2
4. Mesh reinforcement as shown to be A193, continuous across joints and with 50 mm minimum cover to the inner face of wall

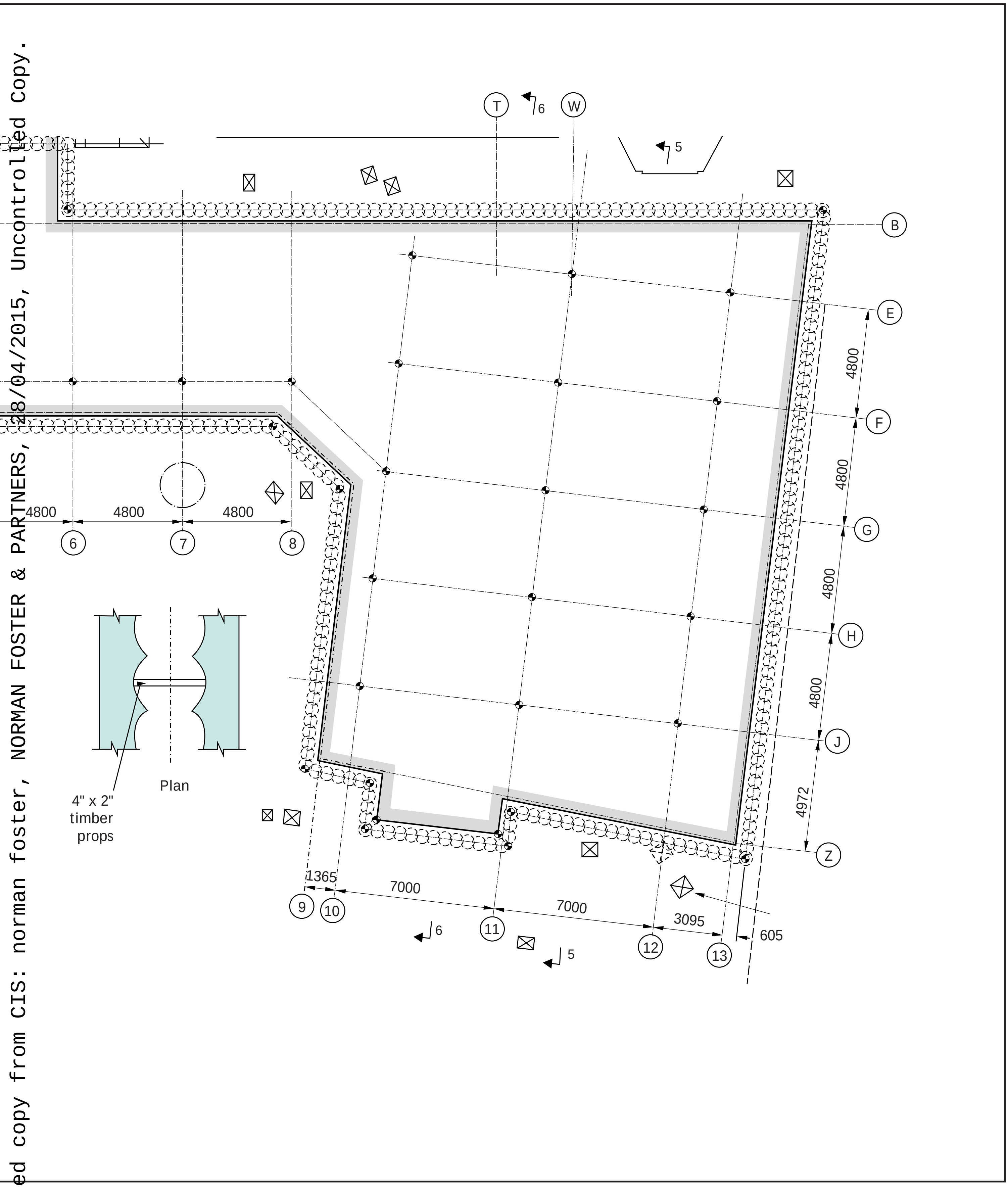
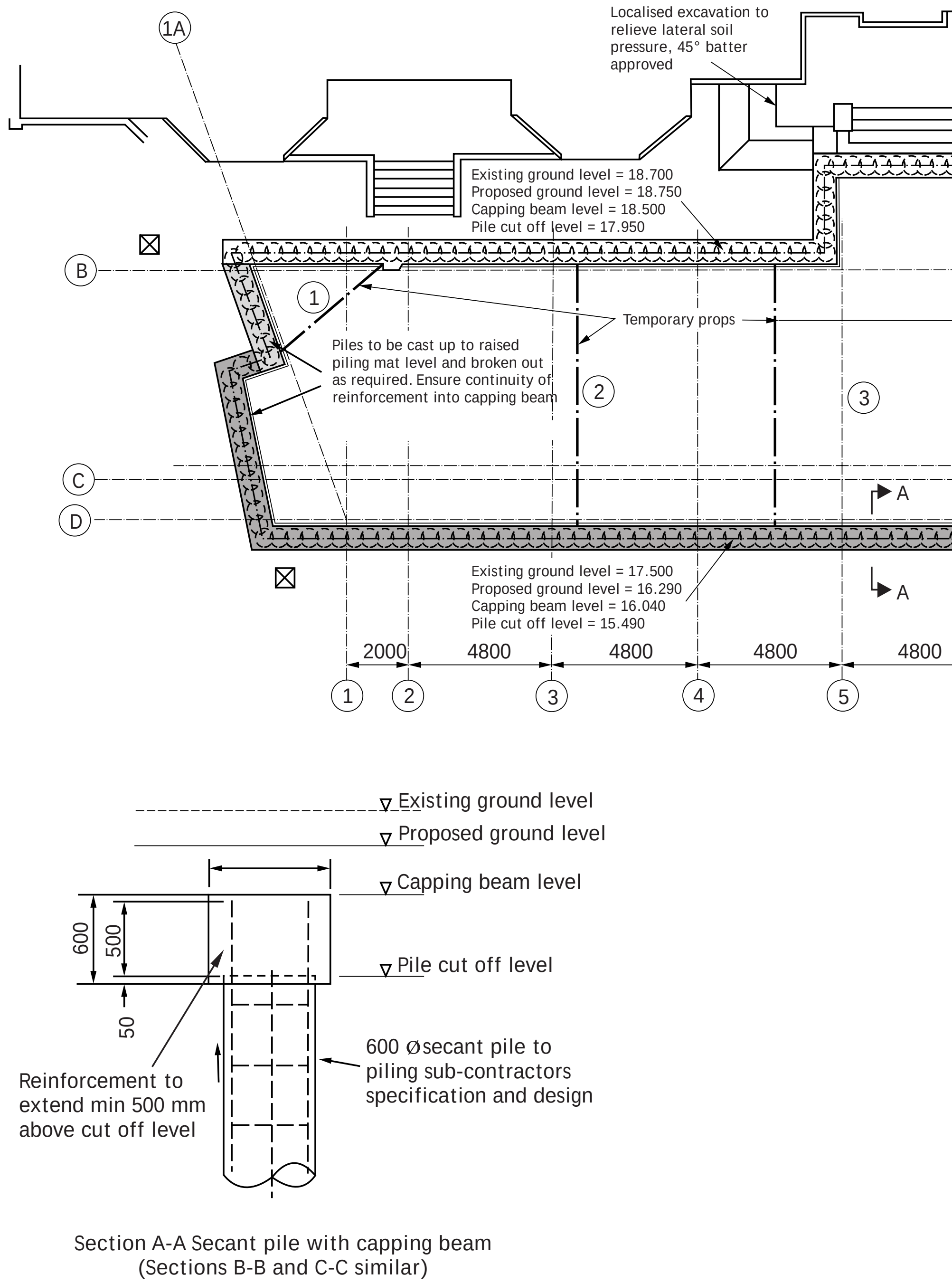

T
6
W
5
B
E
F
G
H
J
Z
4800
4800
4800
4800
4972
4800
4800
4800
6
7
8
4" x 2"
timber
props
Plan
1365
7000
7000
3095
605
9
10
11
12
13
6
5

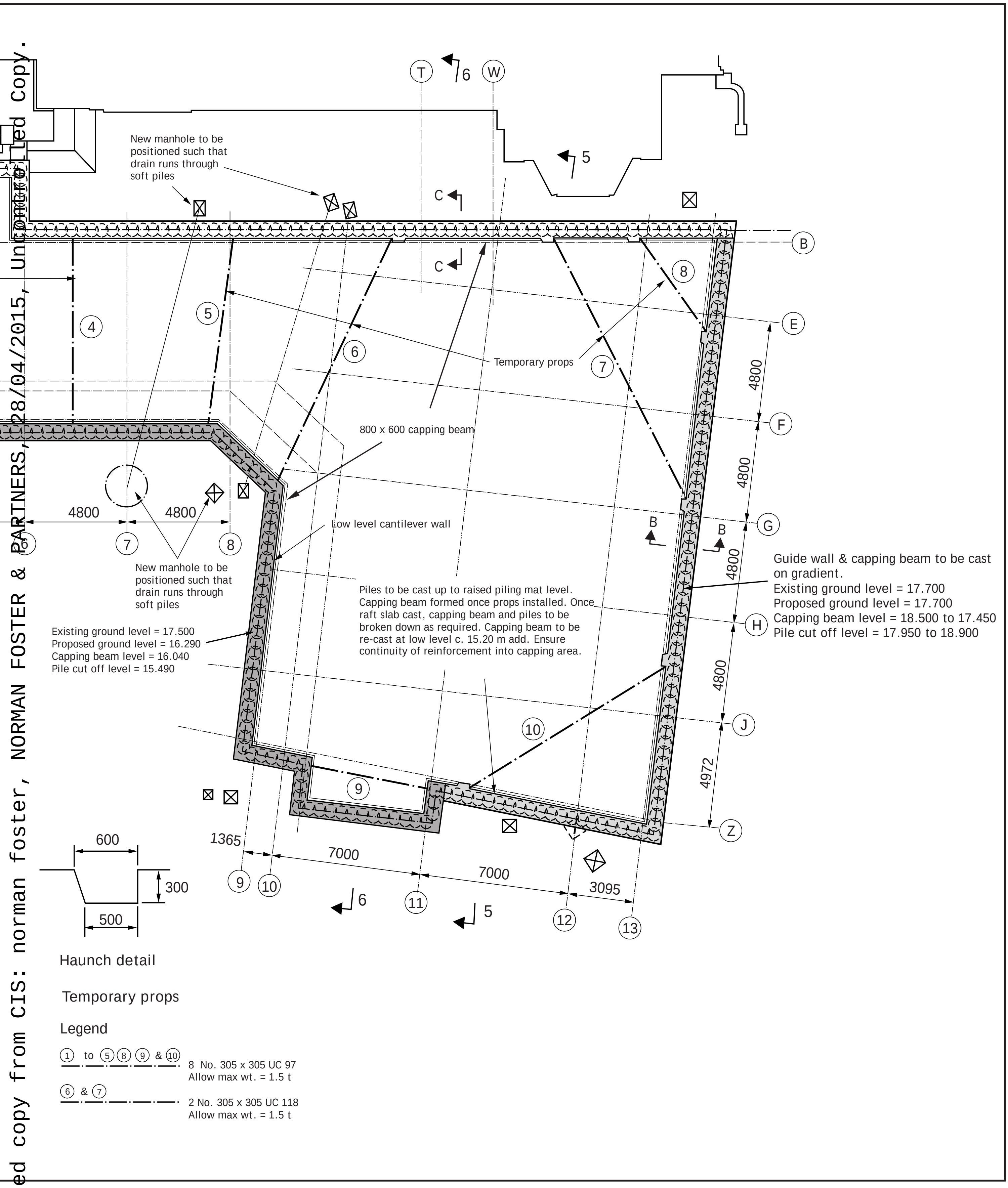
Figure 12.3
Capping beam layout and temporary props.



Section A-A Secant pile with capping beam
(Sections B-B and C-C similar)

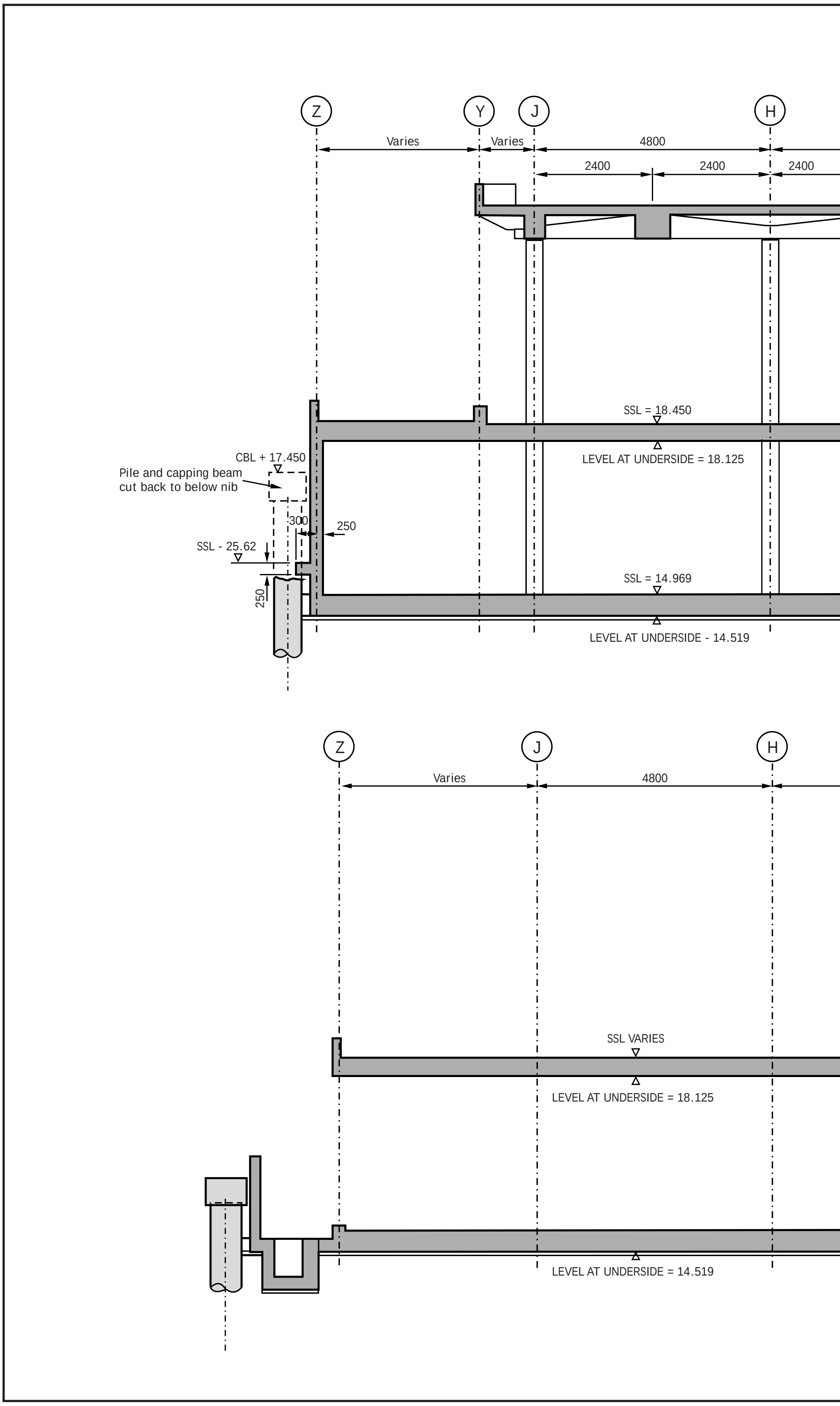
Notes:

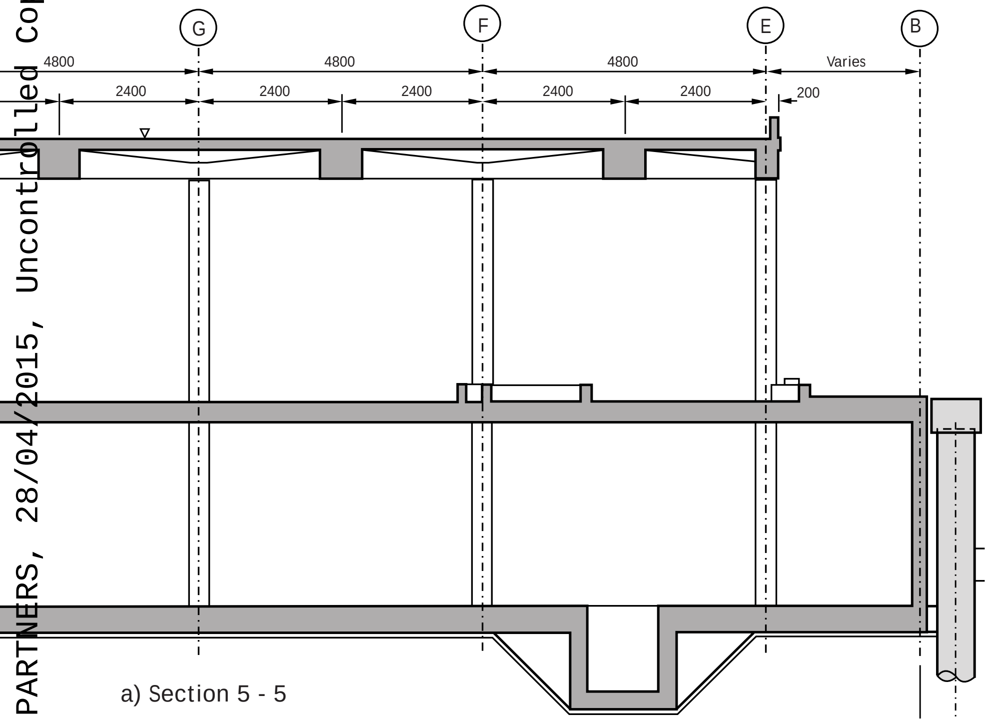
1. For guide wall levels and setting out refer to XYZ - 02
2. Piling mat sequences are to be agreed between contractor and specialist sub-contractor and submitted to the engineer for comment.
3. For piling setting out refer to Drawing XYZ - 03



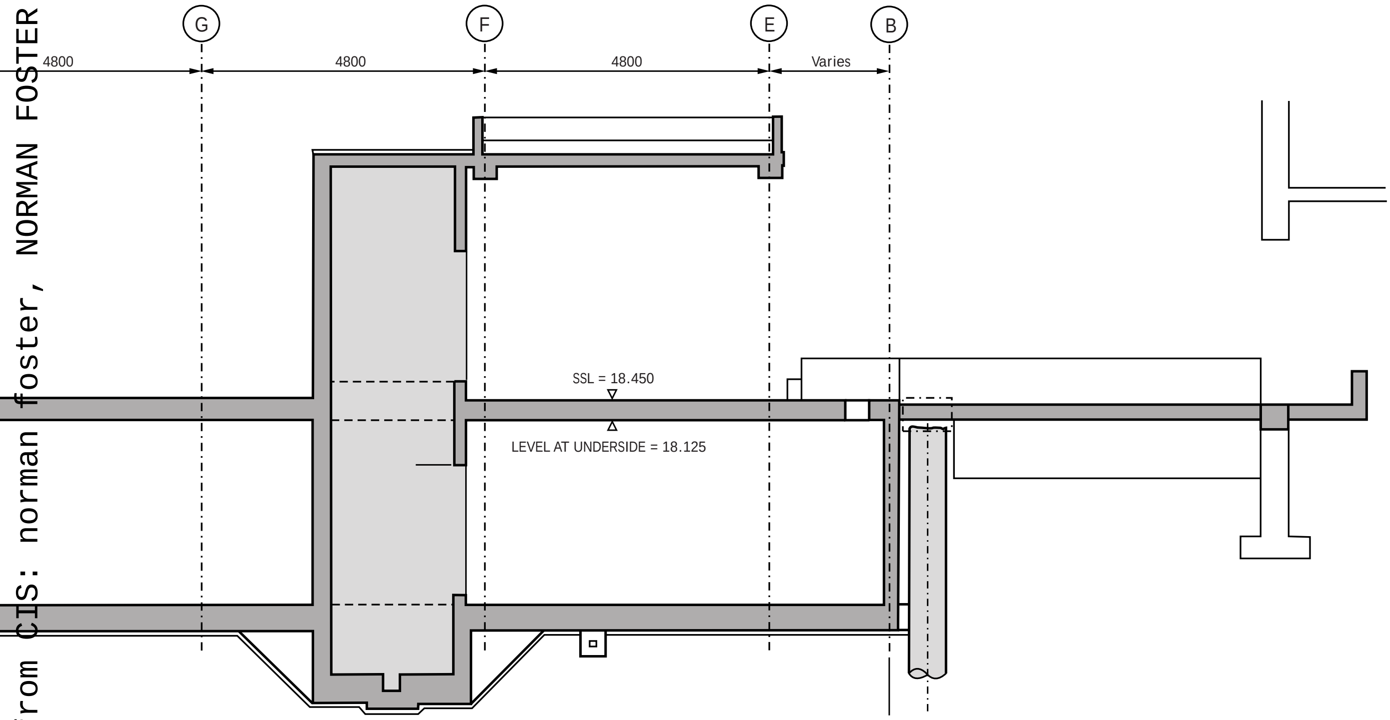
New manhole to be positioned such that drain runs through soft piles
Temporary props
800 x 600 capping beam
Low level cantilever wall
Piles to be cast up to raised piling mat level. Capping beam formed once props installed. Once raft slab cast, capping beam and piles to be broken down as required. Capping beam to be re-cast at low level c. 15.20 m add. Ensure continuity of reinforcement into capping area.
Guide wall & capping beam to be cast on gradient.
Existing ground level = 17.700
Proposed ground level = 17.700
Capping beam level = 18.500 to 17.450
Pile cut off level = 17.950 to 18.900
Existing ground level = 17.500
Proposed ground level = 16.290
Capping beam level = 16.040
Pile cut off level = 15.490
New manhole to be positioned such that drain runs through soft piles
4800
4800
1365
7000
7000
3095
4972
600
300
500
Haunch detail
Temporary props
Legend
1 to 5 8 9 & 10
8 No. 305 x 305 UC 97
Allow max wt. = 1.5 t
6 & 7
2 No. 305 x 305 UC 118
Allow max wt. = 1.5 t

Figure 12.4
Sections 5–5 and 6–6 through basement.





a) Section 5 - 5



b) Section 6 - 6

Figure 12.5
Early excavation.

Figure 12.6
Blinding operation.

Figure 12.7
Pouring the basement slab.

12.2 Case study 2 Basement in South East London

Salient features

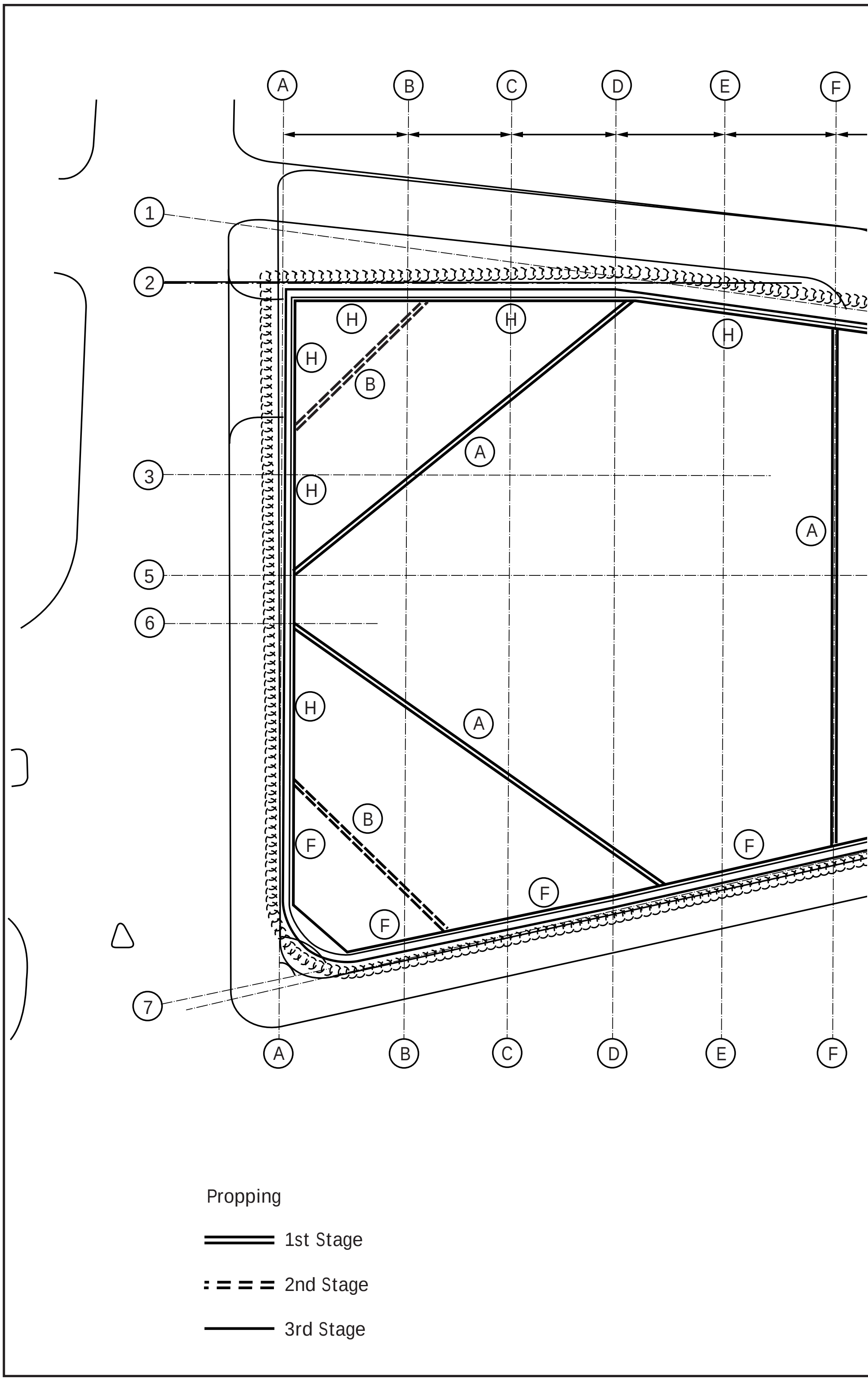
- Two levels of basement (depth of excavation about 7.5 m)
- Use – car park and plant room
- Soil – medium dense gravel below 2.5 m of made ground
- Water table about 4 m below ground level
- Propped sheet pile wall to facilitate excavation
- Reinforced concrete box designed to be inside sheet piled wall

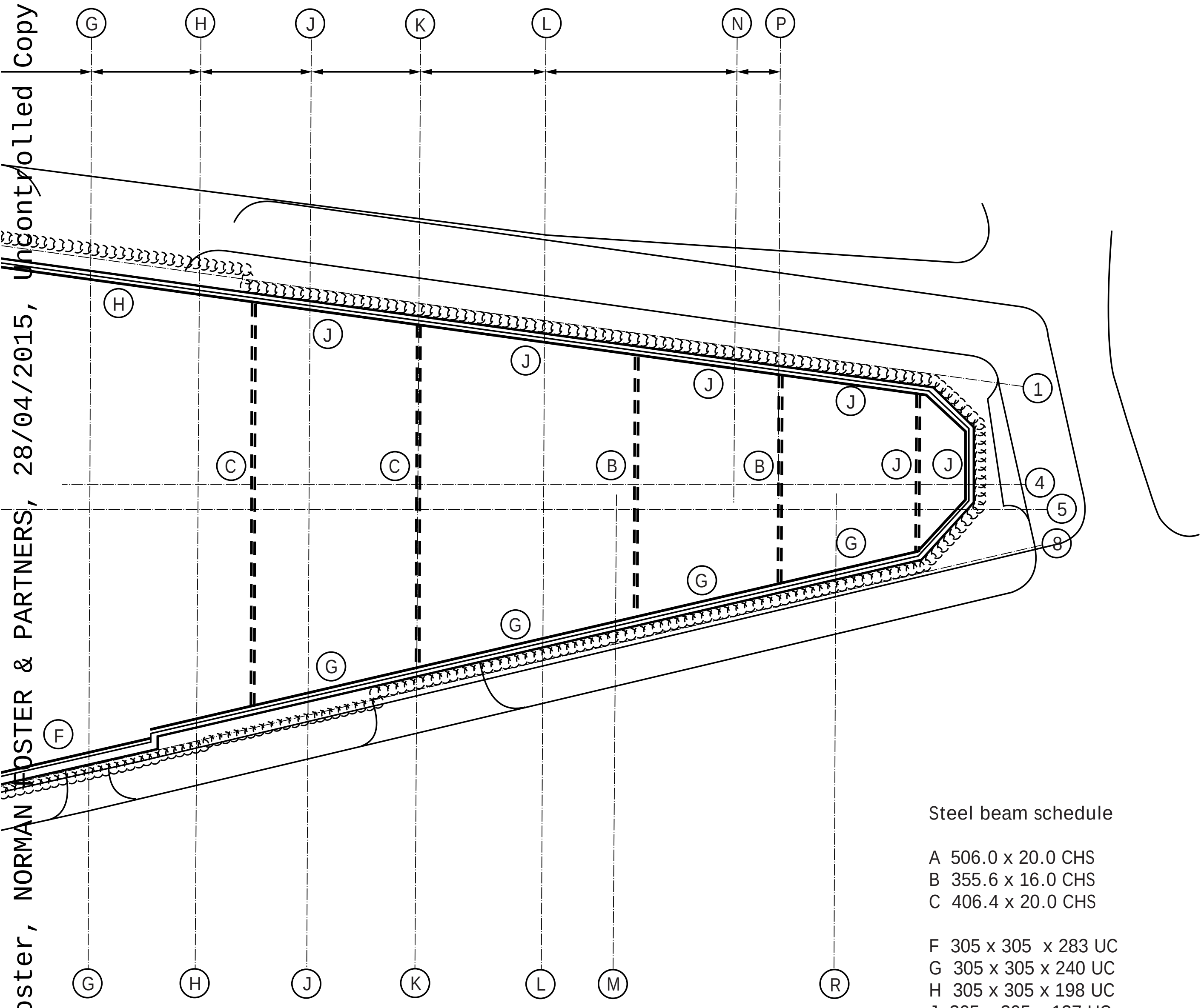
By courtesy Clark Smith Partnership.

Figure 12.8
Temporary propping.
Temporary props to sheet piles remained in place whilst constructing walls on top of the cast basement slab.

Figure 12.9
Ground floor slab under construction.
Props were removed once an adequate propping force was available from the permanent lower basement slab.

Figure 12.10
Temporary beam layout and schedule.





Steel beam schedule

A 506.0 x 20.0 CHS
B 355.6 x 16.0 CHS
C 406.4 x 20.0 CHS

F 305 x 305 x 283 UC
G 305 x 305 x 240 UC
H 305 x 305 x 198 UC
J 305 x 305 x 137 UC

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Appendix A: Design data

A1 Combination factors

Table A1
Combination factors from BS EN 1997-1-1 & NA[11, 11a] and BS EN 1991-4 & NA[55, 55a]

Limit state	BS EN 1997-1-1 & NA							BS EN 1991-4 & NA
	Actions			Materials				Action (liquids)
ULS	γ_G	$\gamma_{G,inf}$	γ_Q	γ_φ	γ_c	γ_{cu}	γ_γ	γ_{Qw}
Combination 1	1.35	1.0	1.5 (0.0)#	1.0	1.0	1.0	1.0	1.2 (0.0)#
Combination 2	1.0	1.0	1.3 (0.0)#	1.25	1.25	1.4	1.0	1.2 (0.0)#
SLS	1.0	1.0	1.0* (0.0)#	1.0	1.0	1.0	1.0	1.0* (0.0)#

Notes:
\# Value if favourable
* For simplicity values of γ_{Qw} =1.0 are advocated. Strictly speaking, the partial factor for variable loads at SLS may be the characteristic combination, frequent or quasi-permanent load. However, in relatively shallow basements the critical limit state is that for cracking where the characteristic combination value is appropriate. ψ_0 may be applied to accompanying variable actions.

A2 Design angle of shearing resistance

Table A2
Design angles of shearing resistance.

Angle of shearing resistance, φ (degrees)

Char	Design #		Char	Design #	
	Combination 1	Combination 2		Combination 1	Combination 2
φ'_k	$\varphi'_{d,Comb\,1}$	$\varphi'_{d,Comb\,2}$	φ'_k	$\varphi'_{d,Comb\,1}$	$\varphi'_{d,Comb\,2}$
20	20.0	16.2	**33**	33.0	27.5
21	21.0	17.1	**34**	34.0	28.4
22	22.0	17.9	**35**	35.0	29.3
23	23.0	18.8	**36**	36.0	30.2
24	24.0	19.6	**37**	37.0	31.1
25	25.0	20.5	**38**	38.0	32.0
26	26.0	21.3	**39**	39.0	32.9
27	27.0	22.2	**40**	40.0	33.9
28	28.0	23.0	**41**	41.0	34.8
29	29.0	23.9	**42**	42.0	35.8
30	30.0	24.8	**43**	43.0	36.7
31	31.0	25.7	**44**	44.0	37.7
32	32.0	26.6	**45**	45.0	38.7

Key
\# to UK NA of BS EN 1997-1-1
φ'_d = tan–1(tan (φ'_k))/ γ_φ)

Note
In granular soils $\varphi'_d \leq \varphi_{crit}$ (see Section 7.3.1)

A3 Pressure coefficients K_{ad} and K_{pd}

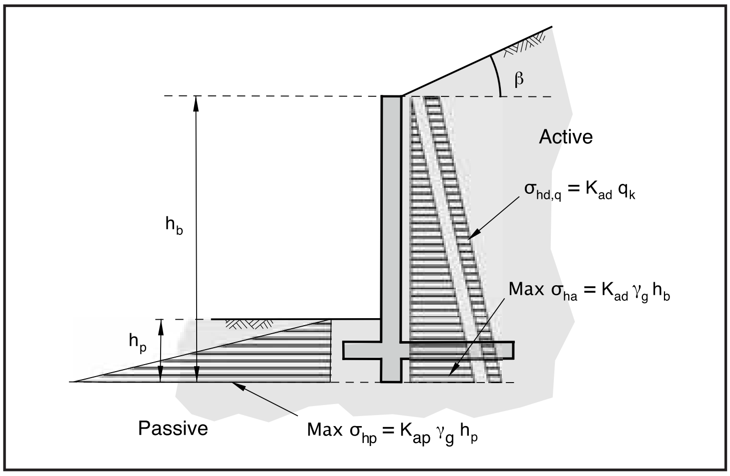


Figure A1
Pressure coefficients K_{ad} and K_{pd}.

Lateral soil pressures on retaining walls may be determined by applying active, at rest or passive pressure coefficients to the effective vertical stress as follows:

Active pressure coefficient[52]

$$K_{ad} = \left[\frac{\cos\beta - (\sin^2\varphi_d - \sin^2\beta)^{0.5}}{\cos\beta + (\sin^2\varphi_d - \sin\beta^2)^{0.5}}\right]\cos\beta$$

Some movement assumed

At rest pressure coefficient

$K_{0d} = 1 - \sin\varphi'_d$	for granular and lightly consolidated soils and	For flat ground surface
$K_{0d} = (1 - \sin\varphi'_d)\,(OCR)^{0.5}$	for over-consolidated soils	For flat ground surface
$K_{0d,}\beta = K_{0d,flat}\,(1 + \sin\beta)$		For sloping sites

Passive pressure coefficient[44]

$$K_{pd} = \left[\frac{\cos\beta + (\sin^2\varphi'_d - \sin\beta^{2})^{0.5}}{\cos\beta - (\sin^2\varphi'_d - \sin\beta^{2})^{0.5}}\right]\cos\beta$$

Requires movement

where

β = inclination of the surface to the horizontal

φ'_d is defined in Section 7.3.1

OCR = over-consolidation ratio for clay soils, which should be determined by tests. In the absence of other data OCR may be taken as 3 as a reasonable value for preliminary design

Table A3
Active, passive and at rest pressure coefficients, K_{ad}, K_{pd} and K_{0d}

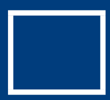
Angle of shearing resistance, φ_d	Angle β (degrees)							
	0		10		20		0	
	K_{ad}	K_{pd}	K_{ad}	K_{pd}	K_{ad}	K_{pd}	K_{0d}[1]	K_{0d}[2]
	Active	Passive	Active	Passive	Active	Passive	At rest	At rest
20	0.490	2.040	0.539	1.854	1.000	1.000	0.658	1.140
21	0.472	2.117	0.517	1.934	0.796	1.257	0.642	1.111
22	0.455	2.198	0.496	2.017	0.720	1.388	0.625	1.083
23	0.438	2.283	0.476	2.103	0.665	1.503	0.609	1.055
24	0.422	2.371	0.456	2.192	0.620	1.612	0.593	1.027
25	0.406	2.464	0.438	2.286	0.582	1.718	0.577	1.000
26	0.390	2.561	0.420	2.383	0.548	1.824	0.562	0.973
27	0.375	2.663	0.403	2.484	0.518	1.931	0.546	0.946
28	0.361	2.770	0.386	2.590	0.490	2.041	0.530	0.919
29	0.347	2.882	0.370	2.702	0.464	2.153	0.515	0.892
30	0.333	3.000	0.355	2.818	0.441	2.269	0.500	0.866
31	0.320	3.125	0.340	2.940	0.419	2.389	0.485	0.840
32	0.307	3.255	0.326	3.069	0.398	2.514	0.470	0.814
33	0.295	3.393	0.312	3.204	0.378	2.644	0.455	0.789
34	0.283	3.538	0.299	3.346	0.360	2.780	0.441	0.763
35	0.271	3.691	0.286	3.496	0.342	2.922	0.426	0.738
36	0.260	3.853	0.274	3.654	0.326	3.071	0.412	0.714
37	0.249	4.024	0.262	3.821	0.310	3.229	0.398	0.690
38	0.238	4.205	0.250	3.998	0.295	3.394	0.384	0.666
39	0.227	4.396	0.239	4.185	0.280	3.569	0.371	0.642
40	0.217	4.600	0.228	4.384	0.266	3.753	0.357	0.619
41	0.208	4.816	0.218	4.594	0.253	3.949	0.344	0.596
42	0.198	5.046	0.208	4.818	0.241	4.156	0.331	0.573
43	0.189	5.291	0.198	5.056	0.229	4.376	0.318	0.551
44	0.180	5.552	0.188	5.310	0.217	4.610	0.305	0.529
45	0.172	5.830	0.179	5.581	0.206	4.860	0.293	0.507

Notes
1 At rest pressure coefficient for granular and lightly consolidated soils.
2 At rest pressure coefficient for overconsolidated soils assuming overconsolidation ratio = 3

A4 Bending moment coefficients for rectangular plates

Tables A4a and A4b may be used to estimate moments in rectangular plates.

Table A4a
Bending moment coefficients for rectangular plates supporting a UDL[71]

Type of panel and moments considered		Short span coefficients, β_z			Long span coefficient, β_x
		Values of k = width l_x/height l_z			
		1	1.5	2	
	**Interior panel**				
	Negative moments at continuous edge	0.031	0.053	0.063	0.032
	Positive moment at mid-span	0.024	0.040	0.048	0.024
	**One long edge discontinuous**				
	Negative moments at continuous edge	0.039	0.073	0.089	0.037
	Positive moment at mid-span	0.03	0.055	0.067	0.028

Note:
Vertical moment = $\beta_z n l_z^2$ Horizontal moment = $\beta_x n l_x^2$
where
l_z = height of panel; l_x = width of panel

Table A4b
Bending moment coefficients for rectangular plates supporting a triangular load[72]

Type of panel and moments considered		Coefficients α_x, α_z				
		Values of k = width l_x/height l_z				
		0.5	1	2	3	4
	**3 sides fixed, top pinned**					
	Negative moments at edge, α_x	0.012	0.029	0.037	0.037	0.037
	Positive moment at mid-span for span l_x, α_x	0.006	0.012	0.01	0.009	0.009
	Negative moments at bottom edge, α_z	0.011	0.035	0.062	0.066	0.067
	Positive moment at mid-span for span l_z, α_z	0.003	0.011	0.026	0.029	0.029
	**3 sides fixed, top free**					
	Negative moments at edge, α	0.012	0.03	0.066	0.091	0.099
	Positive moment at mid-span for span l_x, α	0.006	0.013	0.028	0.024	0.017
	Negative moments at bottom edge, α_z	0.011	0.035	0.086	0.127	0.149
	Positive moment at mid-span for span l_z, α_z	0.003	0.01	0.016	0.011	0.007

Note
Horizontal moment = $\alpha_x n l_z^2$ Vertical moment = $\alpha_z n l_z^2$

A5 Design data for crack width formulae

These data are intended to give an indication of typical values for the various properties required in the determination of crack widths. Values used in design should be verified for the concretes being used and for the specific site circumstances.

A5.1 Mean tensile strengths of concretes f_{ctm} ($\equiv f_{ct,eff}$)

Table A5
Mean tensile strengths of concretes and ρ_{min} $^{\$}$

Strength Class (f_{ck28}/f_{cu28}) MPa	Mean tensile strength of concrete (days), (MPa)							ρ_{min} for sections ≤ 300 mm thick			
	$f_{ctm(3)}$			$f_{ctm(7)}$			$f_{ctm,28}$	t=3days			t=28days
Cement Class#	R	N	S	R	N	S	R, N &S	R	N	S	R, N &S
C20/25	1.47	1.32	1.01	1.81	1.72	1.51	2.21	0.293%	0.264%	0.202%	0.44%
C25/30	1.70	1.53	1.17	2.10	2.00	1.75	2.56	0.340%	0.307%	0.235%	0.51%
C30/37	1.92	1.73	1.33	2.37	2.26	1.98	2.90	0.384%	0.346%	0.265%	0.58%
C40/45	2.13	1.92	1.47	2.63	2.50	2.20	3.21	0.426%	0.384%	0.294%	0.64%
C20/25	2.33	2.10	1.61	2.87	2.73	2.40	3.51	0.465%	0.420%	0.321%	0.70%

Notes
$ Derived from BS EN 1992-1-1
# Numerous types of cement are available and unless specifically stated it is assumed that they do not affect 28 day design properties. However the cement type has a significant effect on the development of concrete strength and other concrete properties. BS EN 1992-1-1 uses Classifications S, N and R (slow, normal and rapid) in expressions to calculate strength gain etc. CEM I cements will be 'R'. CEM II and CEM III, or their equivalents, may be 'S', 'N' or 'R'. More specifically , the Classes are defined in BS EN 1992-1-1 as:

- Class S (cement type CEM 32.5N),
- Class N (cement types CEM 32.5R and CEM 42.5N) and
- Class R (cement types 42.5R, CEM 52.5N and CEM 52.5R).

CEM 32.5N, CEM 32.5R etc. are defined in BS EN 197-1 (Cement)[73]. BS EN 206-1 (Concrete)[12] uses designations CEM I, CEM II, CEM III, etc. Broadly, CEM I cements are Portland cements and will usually be Classification 'R' to BS EN 1992-1-1. CEM II and CEM III, or their equivalents, may be 'S', 'N' or 'R'. However,

- Class N may be assumed where ggbs exceeds 35% of the composition with Portland cement or where fly ash (fa) exceeds 20%,
- Class S may be assumed where ggbs exceeds 65% of the composition with Portland cement or where fly ash (fa) exceeds 35%.
- For typically assumed cement classes see Table A7

At the design stage it is often not clear which Class should be used. Generally in structural works Class R should be assumed. However for water-retaining or water-excluding structures it is usual to assume the specification and use of Class N cements until or unless better information is available. The use of Class R cements leads to greater values of T_1 and the use of Class S may have implications on striking times particularly in cool or cold weather.

A5.2 Coefficient of thermal expansion, α

A conservative value of coefficient of thermal expansion, α_c for aggregates in the UK which should be used in the absence of data is 12×10^{-6}/°C. Where the type of rock group of the coarse aggregate is known and can be guaranteed to be used, the appropriate value from Table A6 may be used e.g. 10×10^{-6}/°C for granites and 9×10^{-6}/°C for limestones.

Table A6
Design value for coefficient of thermal expansion (Based on CIRIA C660[18] Table 4.4 and Table 3 of Properties of Concrete for use in Eurocode 2[74])

Coarse aggregate/rock group	Design value for coefficient of thermal expansion (microstrain/°C)
Chert or flint	12
Quartzite	14
Sandstone	12.5
Marble	7
Siliceous limestone	10.5
Granite	10
Dolerite	9.5
Basalt	10
Limestone	9
Glacial gravel	13
Lytag (coarse and fine)	7

A5.3 Difference between the peak temperature of concrete during hydration and ambient temperature °C, T_1

T_1 is influenced by many factors including cement content, types and sources of cementitious material, cement class (see Note to Table A5), mix proportions, section thickness, formwork and insulation, placing temperature, ambient conditions, formwork removal and possible active forms of temperature control.

For C30/37 concretes, T_1 for walls may be estimated from Table A7. The base assumptions for Table A7 and an indication of the effects of variations to those assumptions are given in Table A8 Further details, guidance and a spreadsheet are included in CIRIA C660[18]

For ground slabs (and suspended slabs) up to 500 mm thick, the value of T_1 may be estimated by taking the value for a wall cast into steel formwork with a thickness of 1.3 × the thickness of the slab[18]. When surface insulation is applied, as a first approximation, T_1 may be assumed to be the same as for a wall of the same thickness cast into ply formwork.

For basements (and water-retaining structures) placing temperature is generally taken as being 20°C and ambient temperature is generally taken to be 15°C.

Formwork system and wall thickness		T_1 for assumed cementitious binder and binder content (°C)									
		CEM I	20% fly ash	30% fly ash	40% fly ash	50% fly ash	not specified (≡ CEM I)	20% ggbs	40% ggbs	60% ggbs	80% ggbs
Assumed cement Class+		R	R	N	S	S	R	R	N	N	S
Indicative binder content* (kg/m³)		340	360	365	380	400	340	340#	340	375	450
Using steel formwork	**250 mm th.**	16	13	11	10	9	16	13	12	10	9
	500 mm th.	28	24	22	20	18	28	24	21	20	17
	750 mm th.	37	32	29	27	24	37	33	30	27	23
	1000 mm th.	43	38	35	32	29	43	40	36	34	29
Using 18 mm ply formwork	**250 mm th.**	24	20	18	16	15	24	21	18	16	14
	500 mm th.	36	32	29	26	24	36	33	29	27	23
	750 mm th.	43	38	35	31	29	43	39	36	34	29
	1000 mm th.	47	42	39	34	33	47	44	41	39	34
Using 37 mm ply formwork	**250 mm th.**	27	23	20	18	16	27	24	20	18	16
	500 mm th.	40	35	32	29	27	40	36	33	31	27
	750 mm th.	45	40	37	34	31	45	42	39	37	32
	1000 mm th.	49	44	40	37	34	49	47	43	41	36

Notes
1 Derived from CIRIA C660[18]: Temperature.xls – Prediction of the early-age temperature rise in concrete.
2 For slabs see text in Section A5.3 above.

Key
* Indicative only and intentionally at high end of the range
\# Assumed
\+ See note to Table A5

Table A7
T_1 for walls assuming C30/37 strength Class concrete.

Table A8
Base assumptions and effect of variations on T_1 for a 375 mm thick wall.

Parameter	Base value		Variation and effect (single variation alone)	
	value	unit	Fall in T_1	Rise in T_1
Binder content	340	kg/m³	– 20 kg/m³ ⇨–1°C#	+ 20 kg/m³ ⇨+ 2°C#
Binder type	CEM 1	%	30% fly ash ⇨–8°C#	
			50% ggbs ⇨–8°C#	
Wall thickness	375	mm	– 50 mm ⇨–2°C	+ 50 mm ⇨+2°C
Formwork type	18	mm ply	Steel ⇨–9°C	37 mm ply ⇨+ 5°C
Density	2400	kg/m³		– 400 kg/m³ ⇨+ 3°C
Specific heat	1	kJ/kg°C		1.05 kJ/kg°C ⇨+ 1°C
Wind speed	4	m/s	16 m/s ⇨–3°C	0 m/s ⇨+ 2°C
Formwork removal	36	hours	12 hrs ⇨–2°C*	72 hrs ⇨+0°C
Thermal conductivity	1.8	W/m°C	2.9 W/m°C ⇨–1°C	1.0 W/m°C ⇨+3°C
Placing temperature	20	°C	@15°C ⇨–5°C	@25°C ⇨+6°C
Ambient temperature (mean)	15	°C	@20°C ⇨–2°C	@10°C ⇨+3°C
Placing time	12.00	hours	no change	
T_1	31	°C		
Maximum temp.	46°C @ 19 hours			
Maximum differential (surface to core)	7°C @ 20 hours			

Notes
Derived from CIRIA C660[18]: Temperature.xls – Prediction of the early-age temperature rise in concrete.
Effects of variations are not cumulative.

Key
not usually equivalent in strength to base concrete.
* not recommended practice. Large temperature differences between the surface and the interior should be avoided. Differences greater than about 20°C may lead to cracking. Forms, insulation or protective measures should be removed gradually to avoid thermal shock.

A5.4 Autogenous shrinkage strain, ε_{ca}

Values of autogenous shrinkage according to BS EN 1992-1-1 Exp. (3.12) are given in Table A9.

Table A9
Values of autogenous shrinkage, ε_{ca}.

Strength Class (f_{ck28}/f_{cu28}) MPa	ε_{ca}(t) microstrain				
	3 days	7 days	14 days	28 days	∞
C20/25	8	11	13	17	25
C30/37	15	21	27	33	50
C40/50	23	32	40	50	75

Note
Autogenous shrinkage is normally considered not to increase beyond 28 days and is deemed to be within drying shrinkage.
ref: BS EN 1992-1-1 Exp (3.12), Exp (3.13)

A5.5 Drying shrinkage strain, ε_{cd}

Using expressions and data from BS EN 1992-1-1, the following tables can be created for drying shrinkage at time 10 000 days:

Table A10
Drying shrinkage at 10 000 days $\varepsilon_{cd,t=10000}$ for C30/37 concretes.

$\varepsilon_{cd,t=10000\ Ext}$, (microstrain)				
	Notional size, h_0, mm			
Cement Class	**200**	**300**	**400**	**500**
	External RH = 85%,			
R	248	217	207	198
N	179	156	150	143
S	144	125	120	114
	Internal RH = 45%,			
R	584	510	488	465
N	421	368	352	336
S	338	296	283	270

Notes
1 Table assumes time at moment considered less time to end of curing $(t - t_s)$ = 10 000 days and two sides exposed. If one side exposed $h_0 = 2 \times$ thickness where h_0 = area of concrete/perimeter.
2 Drying shrinkage is generally taken to be zero at early age.
3 Notional thickness, $h_0 = 2Ac/u$ .Ac = area of concrete section. u =perimeter exposed to drying.
Ref: BS EN 1992-1-1 Exps (3.9), (B.11) and (B.12); Table 3.3

A5.6 Restraint factors, R

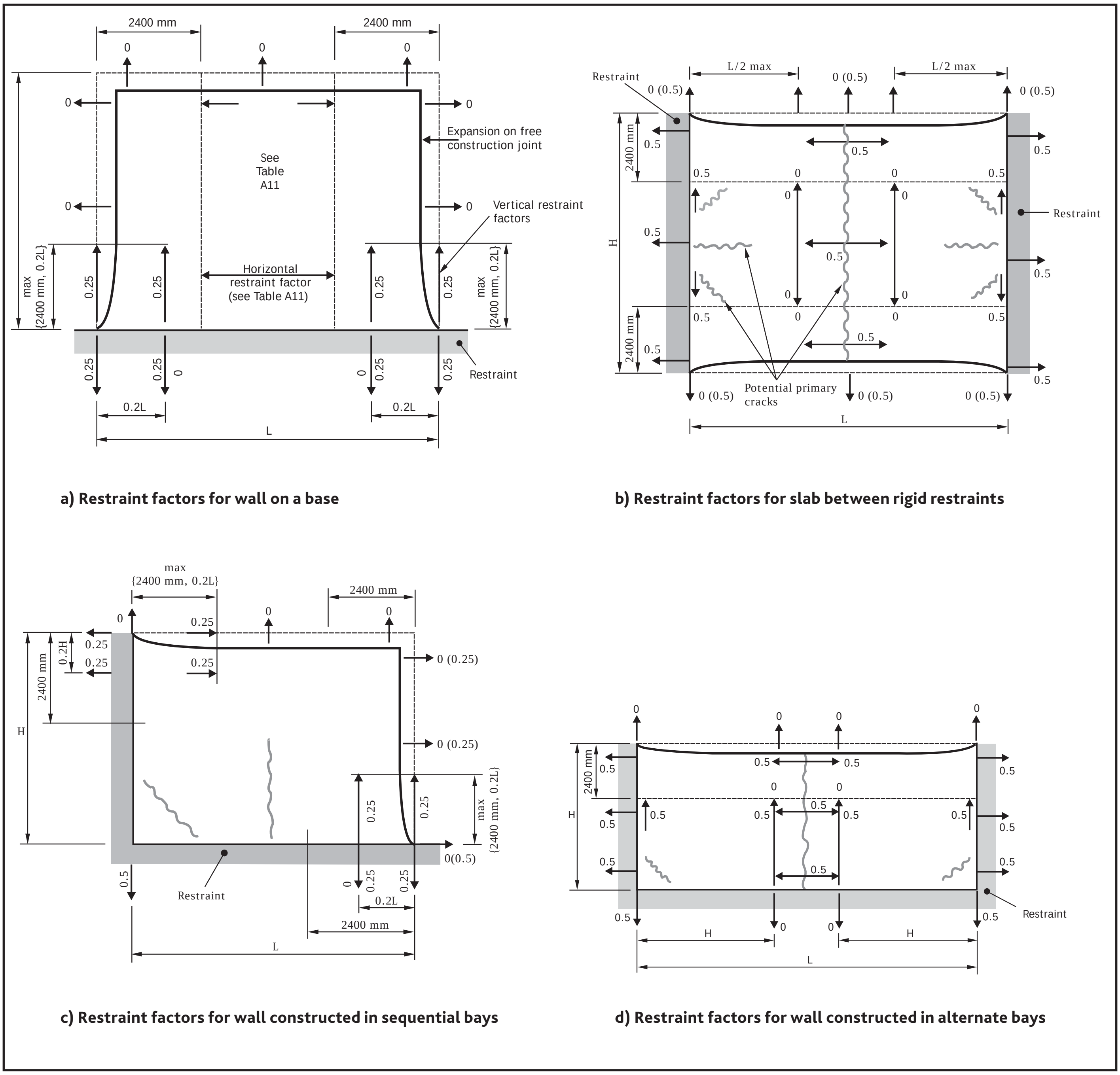
R_1 = the restraint factor appropriate to early age thermal movements both for wall and slabs may be derived from considering restraint factors R below.

R_2, R_3 = appropriate restraint factors for the long term thermal and drying movements respectively. For walls, R_2 and R_3 may be derived from Figure L1 of BS EN 1992-3 (see below) or calculated in accordance with CIRIA C660. See descriptions for factor R below. For slabs, long term restraint factors are usually less onerous.

Restraint factor, R, general

For walls, R may be taken from Figure L1 of BS EN 1992-3 (see Figures A2a to A2d below).

R_{max} = 0.5, which includes a modification factor to account for creep under sustained loading.



a) Restraint factors for wall on a base

b) Restraint factors for slab between rigid restraints

c) Restraint factors for wall constructed in sequential bays

d) Restraint factors for wall constructed in alternate bays

Figure A2
Restraint factors, R

Notes to Figure A2

1. In line with BS 8007 the restraint factors in BS EN 1992-3 include a factor of 0.5 to allow for creep. The factors in Figure A2 should therefore be multiplied by 1.0/0.65 = 1.54 before being used with CIRIA C660 and calculations where the allowance for creep, $K_1 = 0.65$
2. The restraint factors indicated are also appropriate when considering restraint factor R_1 in new pours from adjacent pours in slabs, etc. R_2 and R_3 may be different from R_1. Figures in brackets are restraint factors associated with adjacent construction joints.

Table A11
Horizontal restraint factor for central zone# of walls on a base.

L/H	R at base	R at top of wall
≤2	0.5	0
3		0.05
4		0.3
≥8		0.5

Key
See Figure A2a for location of central zone
Ref: BS EN 1992-3 Table L.1

Restraint factor R alternative [18]

Alternatively for walls cast onto older concrete, R (excluding allowance for creep) may be calculated using CIRIA C660, where restraint factor at the joint, R_j is based on the formula:
$R_j = 1/(E_nA_n/E_oA_o)$

where

E_n (E_o) = Elastic modulus of new (old) concrete. For thin sections $E_n/E_o \approx 0.8$ at 48 hrs[61]

A_n (A_o) = Area of new (old) concrete. For a wall cast at the edge of a slab A_n/A_o may be taken as equal to h_n/h_o. For a wall cast in the middle of a slab A_n/A_o may be taken as equal to $2h_n/h_o$.

The method estimates restraint at the joint and will give values in excess of R = 0.5 for the lower parts of the wall when the section area of new concrete, A_n < 1.43 × section area of old concrete, A_o. See Table A12.

Table A12
Restraint factors R_j near base of wall (excluding effects of creep).

A_n/A_o	0.25	0.50	0.75	1.00	1.25	1.43
$R_{\text{early age}}$	0.85	0.74	0.66	0.59	0.53	0.50
$R_{\text{long term}}$	0.80	0.67	0.57	0.50	0.44	0.41

Notes
1 Based on CIRIA C660[18]
2 Assumes wall cast at edge of slab. For walls cast remote from an edge, restraint factors are higher: the calculated value of A_n/A_o should be divided by a factor of 2.0 to obtain an estimate of R.
3 For a wall cast at the edge of a slab CIRIA C660 recommends assuming that $A_n/A_o = h_n/h_o$

Other restraint factors

Global restraint factors are given in Table A13.

Table A13
Other restraint factors R.

Location	R
Massive pour cast onto blinding	0.1 to 0.2
Massive pour cast onto existing mass concrete (at base)	0.3 to 0.4
Suspended slab (general case)	0.2 to 0.4

Note
Taken from BS 8110-2:1985[58]. Assumed to represent true restraint. i.e. no allowance made for creep[18]

Small domestic basements

The effects of restraint appear to reduce as basements reduce in size. Restraint to movement may be less prevalent In small basement slabs. Small basement walls may not have sufficient length to develop the higher values of restraint factors. Thus the assumption made in Section 9.4 may be considered overcautious for domestic scale basements but applicable restraint factors should be assessed on an individual basis.

Piled slabs

Piles are not always inflexible to lateral loads. For the purposes of estimating whether a slab cracks or not, one method of estimating end restraint of piled slabs is to assume the horizontal movement of a pile to horizontal load equals the vertical movement to vertical load. The force-movement of the piles may then be balanced against the force-movement of the slab to estimate the likely force in the slab and a suitable restraint factor.

A5.7 Tensile strain capacity of the concrete, ε_{ctu}

Values of tensile strain capacity may be taken from Table A14.

Table A14
Tensile strain capacity for sustained loading.

Strength Class (f_{ck28}/f_{cu28}) MPa	$\varepsilon_{ctu(t)}$ (microstrain)			
	ε_{ctu3S} *#	ε_{ctu3N} *#	ε_{ctu3R} *#	ε_{ctu28} *
C20/25	53	63	68	91
C25/30	58	70	75	100
C30/37	63	**76**	81	**108**
C35/45	67	81	87	116
C40/50	71	86	92	123

Key
* Values determined from fctmt/Ecmt according to BS EN 1992-1-1. They include a factor of 1.23 for to allow for the effects of creep and sustained loading (CIRIA C660 Cl. 4.8).
* Values are for quartzite aggregates. For limestone aggregates, add 10%, for sandstone aggregates, add 40% and for basalt aggregates, deduct 20%
# At 3 days, values depend on Class of cement used. See note to Table A5
REF: BS EN 1992-1-1, 3.1.2(6), 3.1.2 (9), Table 3.1

A5.8 Moduli of elasticity of concrete, E_{cm} and modular ratios, α_e

The moduli of elasticity and modular ratios for concretes at 28 days may be taken from Table A15. For other strengths the formula $E_{cm} = 22+ ((f_{ck} + 8)/10)^{0.3}$ (GPa) may be used. The figures in the table are for quartzite aggregates. Where limestone aggregates are used the values should be reduced by 10%, where sandstone aggregates are used by 30% and for where basalt aggregates are used increased by 20%.

Table A15
Moduli of elasticity of concretes, E_{cm} and modular ratios, α_e

Strength Class (f_{ck28}/f_{cu28}) MPa	E_{cm} GPa	α_e ($\phi = 0$)	α_e ($\phi = 1$)	α_e ($\phi = 2$)	α_e ($\phi = 3$)
C20/25	30.0	6.7	13.4	20.0	26.7
C25/30	31.5	6.4	12.7	19.1	25.4
C30/37	32.8	6.1	12.2	18.3	24.4
C35/45	34.1	5.9	11.7	17.6	23.5
C40/50	35.2	5.7	11.4	17.0	22.7

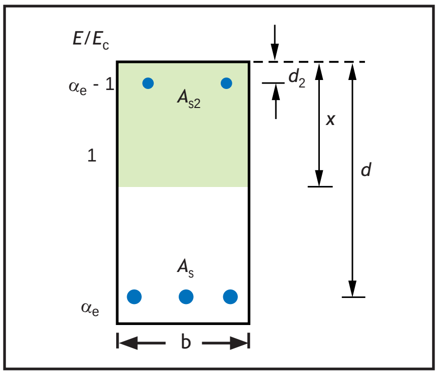
Note:
For the recommended design values for α_e see Section 9.7.4.

Appendix B: Neutral axes and SLS stresses

B1 Neutral axis at SLS (cracked section and no axial stress)

To determine x, neutral axis depth, and inflexure for a cracked concrete section, at SLS, consider the beam in Figure B1:

Figure B1
Cracked concrete section at SLS.



From first principles, for a fully cracked transformed section,

Total area of section, A $= bx + A_s\alpha_e + A_{s2}(\alpha_e - 1)$

1st moment of area, Ay $= bx^2/2 + A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)$

For a slab, $b = 1000$, therefore

A $= 1000x + A_s\alpha_e + A_{s2}(\alpha_e - 1)$

1st moment of area Ay $= 500x^2 + A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)$

Neutral axis depth x $= Ay/A$

$= [500x^2 + A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)]/[1000x + A_s\alpha_e + A_{s2}(\alpha_e - 1)]$

Therefore,

$x[1000x + A_s\alpha_e + A_{s2}(\alpha_e - 1)]$ $= [500x^2 + A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)]$

0 $= [500x^2 + A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)] - x[1000x + A_s\alpha_e + A_{s2}(\alpha_e - 1)]$

$= 500x^2 - x[1000x] + A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)] - x[A_s\alpha_e + A_{s2}(\alpha_e - 1)]$

$= -500x^2 - x[A_s\alpha_e + A_{s2}(\alpha_e - 1)] + [A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)]$

Solving the quadratic,

x $= [-b +/- (b^2 - 4ac)^{0.5}]/2a$

$= -[A_s\alpha_e + A_{s2}(\alpha_e - 1) \pm \{[A_s\alpha_e + A_{s2}(\alpha_e - 1)]^2 + 4 \,.\, 500\, [A_s d\alpha_e + A_{s2}d_2(\alpha_e - 1)]\}^{0.5}]/(2 \times 500)$

or transposing,

x $= [-(\alpha_e - 1)A_{s2} - \alpha_e A_s + \sqrt{}\{[(\alpha_e - 1)A_{s2} + \alpha_e A_s]^2 + 2000[(\alpha_e - 1)A_{s2}d_2 + \alpha_e A_s d]\}]/1000$

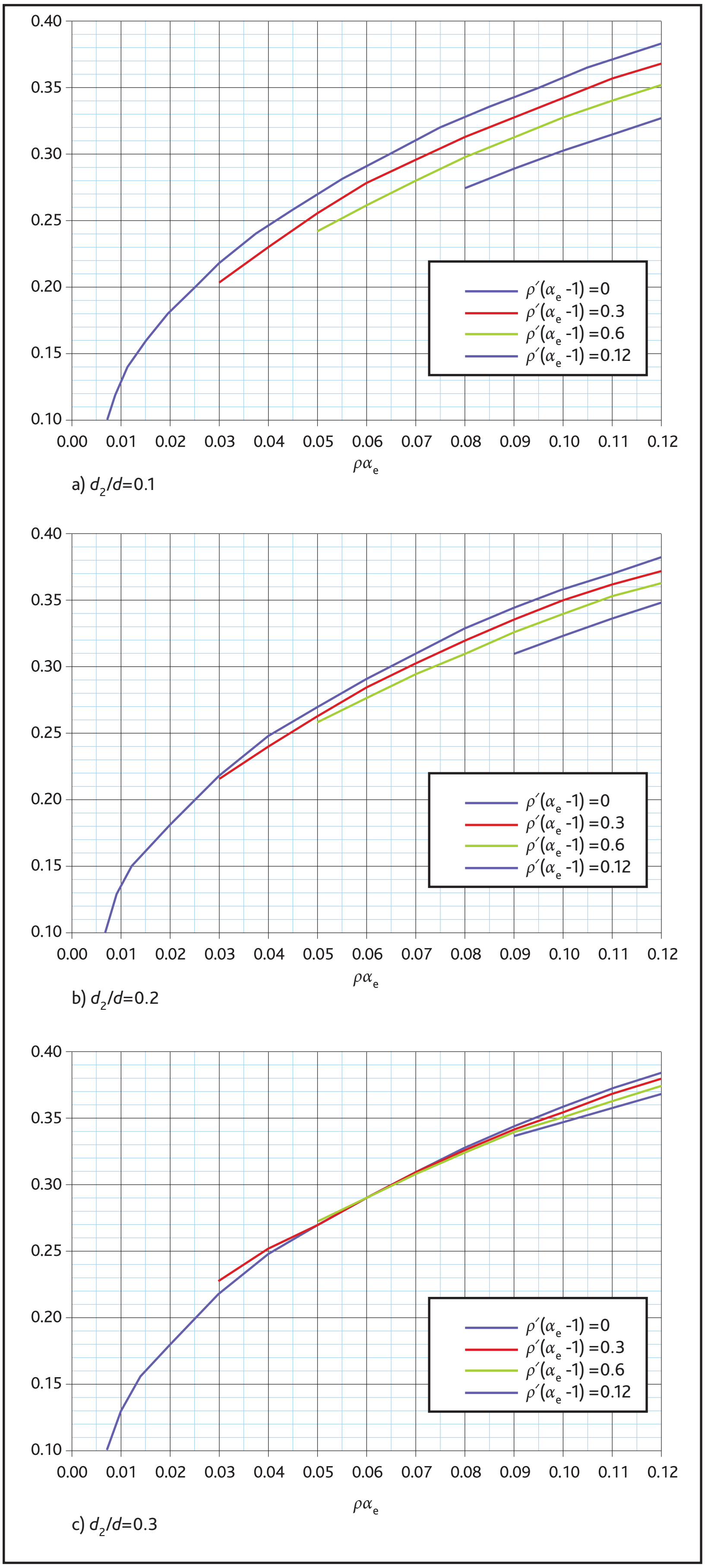
or

x $= [-(\alpha_e - 1)A_{s2} - \alpha_e A_s + \sqrt{}\{[(\alpha_e - 1)A_{s2} + \alpha_e A_s]^2 + 2b[(\alpha_e - 1)A_{s2}d_2 + \alpha_e A_s d]\}]/b$

(as used in RC Spreadsheets[61])

x/d may also be determined from charts such as Figure B2.

Figure B2
Neutral axis depth and lever arm factors for transformed rectangular sections[76]



Note
For cracked sections with no axial stress.

B2 SLS stresses in concrete, σ_c and reinforcement, σ_s (cracked section and no axial stress).

B2.1 Singly reinforced section

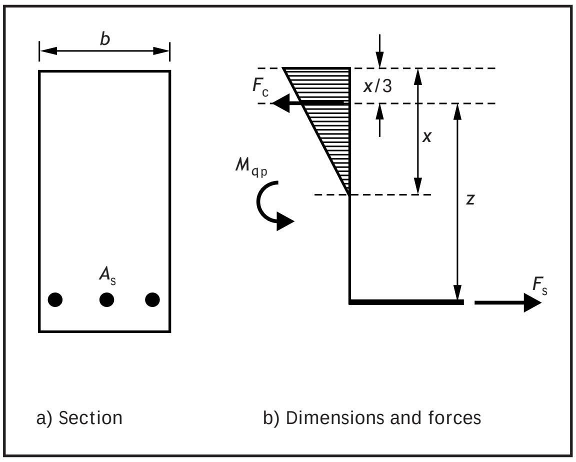


Figure B3
SLS stresses: singly reinforced cracked section.

Consider moments about F_c:

$$M_{qp} = F_s z = F_s(d - x/3)$$
$$F_s = M_{qp}/(d - x/3)$$
$$\sigma_s = M_{qp}/[A_s(d - x/3)]$$
$$\sigma_s A_s = M_{qp}/\,(d - x/3) = xb\sigma_c/2$$
$$\sigma_c = 2\sigma_s A_s/xb$$

B2.2 Doubly reinforced section

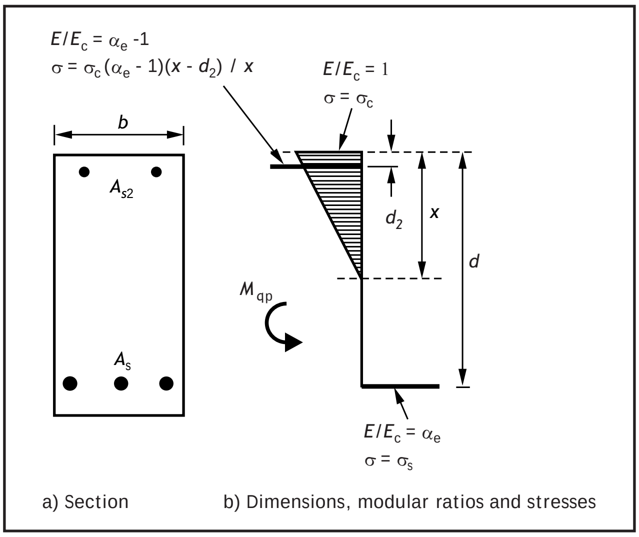


Figure B4
SLS stresses: doubly reinforced cracked section.

Consider moments about A_s[76]

$M_{qp} = A_{s2}(d-d_2)(\alpha_e-1)\{(x-d_2)/x\}\sigma_c + \sigma_c b(x/2)(d-x/3)$

Therefore

$\sigma_c = M_{qp}/[As_2(d-d_2)(\alpha_e-1)\ )\{(x-d_2)/x\} + b(x/2)(d-x/3)]$

Consider moments about A_{s2}:

$M_{qp} = A_s\sigma_s(d-d_2) - b\,\sigma_c\,(x/2)\,(x/3-d_2)$

Therefore

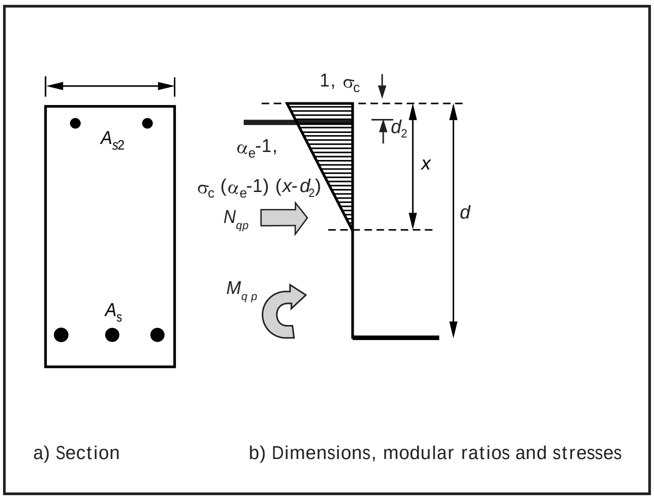
$\sigma_2 = [M_{qp} + b\,\sigma_c\,(x/2)\,(x/3-d_2)]/A_s(d-d_2)$

or more simply from stress diagram

$\sigma_s = \sigma_c\,\alpha_e\,(d-x)/x$

B3 SLS stresses in concrete, σ_s due to flexure and axial load

Figure B5
SLS stresses: doubly reinforced cracked section with axial load.



When an axial force is present, determination of the neutral axis depth has to be via a far more complex method as below.

Let N_{qp} = axial force (compression +'ve)

$F_c = \sigma_c bx/2$

$F_{sc} = (\alpha_e-1)A_{s2}\sigma_c(x-d_2)/x$

$F_{st} = \alpha_e A_s \sigma_c(d-x)/x$

Summing forces,

$F_c + F_{sc} - F_{st} - N_{qp} = 0$

$\sigma_c bx/2 + (\alpha_e-1)A_{s2}\sigma_c(x-d_2)/x - \alpha_e A_s\sigma_c(d-x)/x - N_{qp} = 0 \qquad (1)$

Taking moments about A_s

$F_c(d\text{-}x/3) + F_{sc}(d-d_2) - N_{qp}\,(d-h/2) = M_{qp}$

$\sigma_c b(x/2)(d\text{-}x/3)\ + (\alpha_e-1)A_{s2}\sigma_c(x-d_2)(d-d_2)/x - N_{qp}\,(d-h/2) - M_{qp} = 0 \qquad (2)$

x and σ_c can then be found by solving (1) and (2) as simultaneous equations. Strains, ε, may be then determined.

However, if σ_c is greater than $0.45f_{ck}$, creep becomes non-linear so φ and thus α_e have to be modified according to EC2 expression (3.7). In this range, x and σ_c can only be found by iteration. If σ_c is greater than $0.6f_{ck}$, the stress block starts to become plastic and the section thickness should be increased.

For a more detailed consideration of crack widths due to combined bending and tension see Kruger and Atkinson[77].

Concrete Basements

This guide covers the design and construction of reinforced concrete basements and is in accordance with the Eurocodes.

The aim of this guide is to assist designers of concrete basements of modest depth, i.e. not exceeding 10 metres. It will also prove relevant to designers of other underground structures. It brings together in one publication the salient features for the design and construction of such water-resisting structures.

The guide has been written for generalist structural engineers who have a basic understanding of soil mechanics.

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